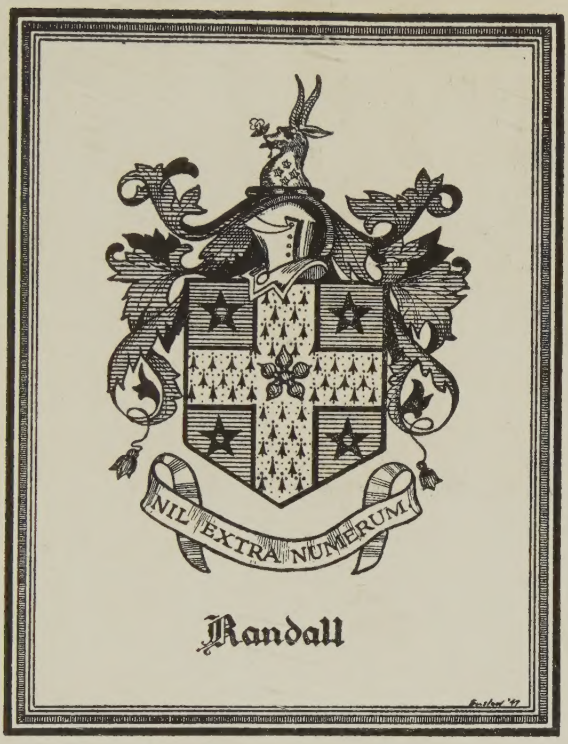


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AMERICAN CONCRETE INSTITUTE

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Memorandum On Arch Dam Developments

BY LARS JORGENSEN*

SYNOPSIS

The intention in the presentation of this paper is to record some of the recent developments in arch dam design and construction in general and in Constant Angle Arch Dam design in particular.

The work described covers the period of the last 16 years. During this period more arch dams have been built than ever before and their design has finally been developed upon as scientific a foundation as that of any other type of dam.

The possibility of obtaining a high factor of safety economically is beginning to become generally recognized by the profession. This was not so 16 years ago.

Actual cost data have been given for the various structures described—covering all kinds of locations—in order to have reliable figures available as a guide for estimating the cost of new work.

Arch Dams can not be called new and untried any longer since the highest dams built and being built are now of that type.

As time goes by, the art of dam building is improving along with other lines of activities. Faith in arch dams is beginning to manifest itself among engineers, especially among the younger set; and the preference shown by many engineers for gravity dams is beginning to give place to a desire for a more economical structure such as an arch dam on sites where this type fits.

An engineering structure must first of all be safe, and secondly it should be economical. It does not require much skill merely to use plenty of material in a structure in order to make it safe; the material should always be placed where it does the most good. The factor of safety may properly vary on various structures, depending upon their importance, but this does not alter the fact

*Consulting Hydro-Electric Engineer, San Francisco.

that whatever material is used, it should be placed as economically as possible.

In a gravity dam it is not possible to utilize the material to the best advantage; half of it is not fully working with reservoir full. Nevertheless, there are cases where this type still remains the best type to use.

In the past, gravity dams have been almost exclusively built on large important projects. At present, however, the arch dam theory has been developed to a stage where it is possible to predetermine the stresses in such a structure with a satisfactory degree of dependability, and with as great accuracy as the stresses in a gravity dam near the foundation. There is no more uncertainty about the behavior of an arch dam than about most any other structure. Enough arch dams have been built and are building to inspire confidence that the principle is right. About the only mystery about arch dams is that no single arch dam has ever failed; there are several freaks existing, almost mysteriously holding a reservoir full of water. (Crowley Creek, Bear Valley Upper Otay.)

Professor Cain put the calculation of arch dams on a better technical basis than it had been before the appearance of his paper "The Circular Arch Under Normal Load."* If his method of calculation is followed, using conservative stresses, a structure of definite safety will result. The lower the stresses are assumed naturally the higher the factor of safety will be. How high it should be depends, as already stated, upon the importance of the work.

Professor Cain's theory as well as the work of Jakobsen, Noetzli and others is to a great extent responsible for the conversion of many members of the profession from being blind gravity enthusiasts to arch dam considerers or even to arch proponents.

There is hardly a subject to be found in engineering where two factions of engineers have been opposed to one another so definitely, as on the subject of gravity dams versus arch dams. On sites where arch dams do not fit, of course, they have no place. In this connection, the words "do not fit" mean where they are not safer and cheaper.

The fact that no single arch dam has ever failed and that to the

*Trans. Am. Soc. Civil Engineers, 1922, page 233.

writer's knowledge 25 gravity dams have failed from one cause or another (19 during the last 30 years),† should be convincing proof that even the most conservative engineers cannot afford to overlook the possibility of the arch dam. The engineer is responsible to his client and should give him the best for his money, regardless of the inconvenience it may cause the engineer to investigate a field new to him.

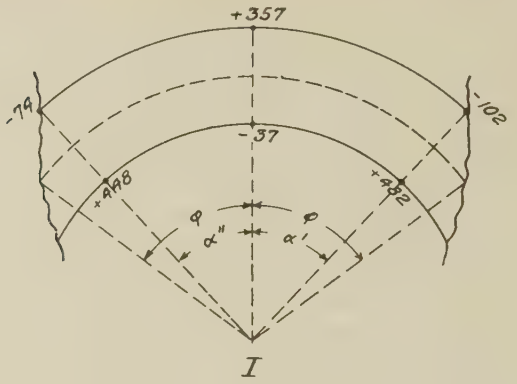
To choose a gravity dam in any and all cases merely because the engineer has been an advocate of this type of dam all his life is a poor foundation for a choice. A decision should be based upon the results of calculations of safety and cost of structure, and not upon preconceived or inherited ideas.

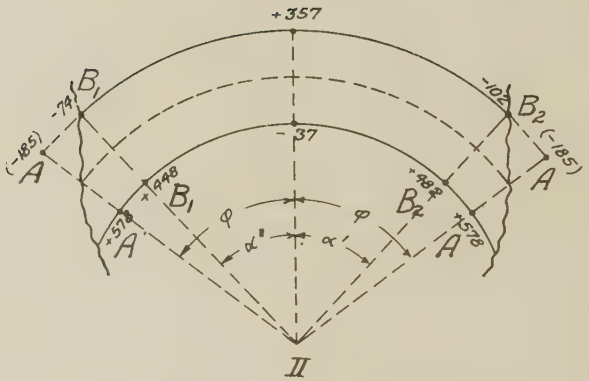
It has been mentioned above that Cain's formulas for a fixed arch under normal load can be regarded as standard; they are now used in design work done by the Constant Angle Arch Dam Co., San Francisco, with which the writer is connected. Some difference of opinion may exist as to how to apply the formulas in cases where the rock abutments are not radial, and such cases are common.

The three sketches, Figures 1, 2 and 3, illustrate various assumptions possible as to span, length, subtended angle, etc., when the abutments are not radial. When calculating the bending moment at the crown the subtended angle of $2\ \phi$ is used in Fig. 1. This angle subtends the points of intersection of the middle line with the abutments. The middle line is assumed to be equal and generally is equal to the average length of the arch, subject to the elastic deformation. The plane of maximum bending moment is taken to be where a radial line first intersects the rock abutments, either on the up or downstream face, as the case may be. Here the water load is assumed to stop on the arch proper. When calculating the maximum bending moment here the smaller value $2\ \dot{\alpha}$ is used for the subtended angle.

For a thick arch, additional calculations are made assuming the maximum plane of bending to be along the plane A-A, Fig. 2, being a radial plane through the point of intersection of the center line or middle line of the arch ring with the rock abutment. The subtended angle $2\ \phi$ is used for the calculation of the bending

†See "Record of 100 Dam Failures," by Lars Jorgensen, Journal of Electricity, San Francisco, March 15, April 1, 1920.





FIGS. 1 AND 2—PACIOMA DAM. STRESSES IN POUNDS PER SQUARE INCH, ELEVATION 1825

moments both at the crown and at the abutments, A-A in this case.

This gives a check on unit compression at the downstream face in plane A-A. If found too high, the dimension should be increased. No attention is paid to possible tension in the rock along the continuation of the upstream face, as none can actually exist.

In Fig. 3, the third possible assumption as to the location of the planes of maximum bending moments and value of subtended angle 2 ϕ is shown. Here the same subtended angle 2 ϕ is used

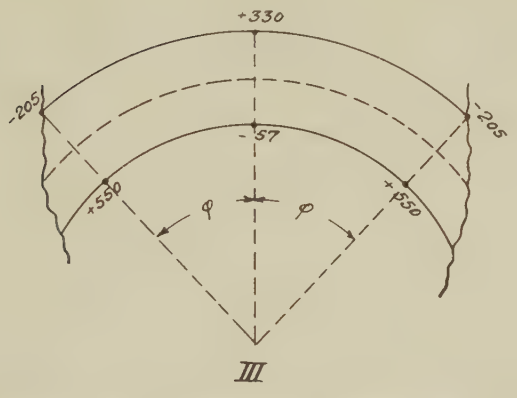


FIG. 3—PACOIMA DAM. STRESSES IN POUNDS PER SQUARE INCH, ELEVATION 1825

for the calculation of both the bending moments at the crown and at the abutments. The influence of using a smaller subtended angle on these bending moments is to the effect of generally lowering the stresses at the crown, at least at the upstream face, and increasing them at the abutments. The stresses at the abutments practically always control the design. On important work it may be well to use all three variations for checking purposes.

When making a design for an arch dam, the simple arch formula (*unit water pressure* $\times$ *length of upstream radius* = *unit stress* $\times$ *thickness*) can most conveniently be used in order to obtain a preliminary section. Then the bending moments and resulting stresses are calculated by means of Cain's formulas. Since the bending moments at the abutments are approximately twice as high as the bending moment at the crown, the thickness at the abutments will not be the same as the thickness at the crown, but will have to be increased towards the abutments to correspond to the increase in stress due to the effect of the larger bending moment. This increase in thickness should commence at a point located between 15 and 18 per cent of the length of the arch out from each abutment. The addition of material should be placed mostly on the downstream face and should increase in thickness from 0 at $\frac{15}{100}$ to $\frac{18}{100}$ of length of arc from the abutments, to a maximum at the abutments.

TABLE TO ACCOMPANY FIGURES 1, 2 AND 3

			Elev. 1825 I	Elev. 1825 II	Elev. 1825 III
t_o thickness of dam at crown		ft.	51.0	51.0	51.0
t_1 thickness of dam at abutment		ft.	51.0	51.0	51.0
R_u		ft.	160.0	160.0	160.0
$r = R_u - \frac{t_o}{2}$		ft.	134.5	134.5	134.5
r_2		ft.²	18090	18090	18090
$2\phi°$			95.0°	95.0°	85.5°
$\phi°$			47.5°	47.5°	42.75°
$\phi = \phi° \times \frac{\pi}{180} = \phi \times 0.01745$			0.82888	0.82888	0.74599
$\sin 2\phi$			0.99619	0.99619	0.99692
$\sin \phi$			0.73728	0.73728	0.67880
$\cos \phi$			0.67559	0.67559	0.73432
$\frac{\sin \phi}{\phi}$			0.88949	0.88949	0.90993
$1 - \frac{\sin \phi}{\phi}$			0.11051	0.11051	0.09007
α'	(right abutment)		43.5°	43.5°	—
α''	(left abutment)		42.0°	42.0°	—
$\cos \alpha'$			0.72537	0.72537	—
$\cos \alpha''$			0.74314	0.74314	—
$k^2 = \frac{1}{12} \times t_o^2$			216.75	216.75	216.75
$D = \left(1 + \frac{k^2}{r^2}\right) \times \phi \times (\phi + \frac{1}{2} \sin 2\phi) - 2 \sin^2 \phi$			0.02594	0.02594	0.01794
$D^s = D + 2.88 \times \frac{k^2}{r^2} \times \phi \times (\phi - \frac{1}{2} \sin 2\phi)$			0.03540	0.03540	0.02431
$\frac{D}{D^s}$			0.7328	0.7328	0.7380
$\frac{2 \sin \phi}{D^s}$			41.654	41.654	55.845
p lbs. pr. sq. ft.			12,500	12,500	12,500
$p_1 = p \times \frac{R_u}{r}$ lbs. pr. sq. ft.			14,870	14,870	14,870
$p_1{}^r$			2,000,000	2,000,000	2,000,000
$(p_1{}^r - P_o)^s = 2\frac{\sin}{D^s} \phi \times p_1 \times \phi \times \frac{k^2}{r}$			827,363	827,363	998,309
Crown	$P_o{}^s$	lbs.	1,172,637	1,172,637	1,001,691
Crown	$M_o{}^s = -(p_1{}^r - P_o)^s \times r \times \left(1 - \frac{\sin \phi}{\phi}\right)$	lbs. ft.	12,297,588	12,297,588	12,093,930
Abutment I	$P_1{}^s = p_1{}^r - (p_1{}^r - P_o)^s \cos \alpha'$ (right abutment)	lbs.	1,399,856	—	—
Abutment I	$M_1{}^s = (p_1{}^r - P_o)^s \times r \times \left(\frac{\sin \phi}{\phi} - \cos \alpha'\right)$ (right abutment)	lbs. ft.	18,263,326	—	—
Abutment I	$P_1{}^s = p_1{}^r - (p_1{}^r - P_o)^s \cos \alpha''$ (left abutment)	lbs.	1,385,153	—	—
Abutment I	$M_1{}^s = (p_1{}^r - P_o)^s \times r \left(\frac{\sin \phi}{\phi} - \cos \alpha''\right)$ (left abutment)	lbs. ft.	16,285,875	—	—
Abutment II	$P_1{}^s = p_1{}^r - (p_1{}^r - P_o)^s \cos \phi$	lbs.	—	1,441,042	—
Abutment II	$M_1{}^s = (p_1{}^r - P_o)^s \times r \times \left(\frac{\sin \phi}{\phi} - \cos \phi\right)$	lbs. ft.	—	23,802,861	—
Abutment III	$P_1{}^s = p_1{}^r - (p_1{}^r - P_o)^s \cos \phi$	lbs.	—	—	1,266,922
Abutment III	$M_1{}^s = (p_1{}^r - P_o)^s \times r \times \left(\frac{\sin \phi}{\phi} - \cos \phi\right)$	lbs. ft.	—	—	23,579,604

STRESSES IN LBS. PER SQ. INCH, ELEV. 1825

Towards the crest of the dam where excess material is used to provide for a practical thickness the up and downstream faces can be kept concentric. Towards the bottom the shear action predominates, especially in narrow canyons, and the dam is to be investigated for this action in its lower portion.

ALLOWABLE STRESSES

When calculating the bending moments and resulting stresses on actual structures, as shown later, the normal load on the structure is taken to equal the full water pressure; the portion of the water load taken by the cantilever has been neglected. To find the portion of load taken by the cantilever is a difficult problem; the amount taken in the middle of the canyon differs from the amount taken at other points and due to the effect of water soaking the cantilever may not take any load. Therefore, this action has been neglected, which is on the side of safety.

Before deciding upon the maximum stress to use, the ultimate strength of the concrete must be determined and the importance of the structure considered.

The writer first used a concrete mix which would have a crushing strength of 1500 to 2000 lbs. per sq. in. when 28 days old. Recently only the upper limit and above has been considered. The amount of cement is varied in such a manner as to give this strength when mixed with the aggregate at hand at the different locations. Less than a barrel of cement to the cubic yard is never used and more than a barrel of cement per cubic yard of concrete often has to be used in order to bring the strength and water-tightness up to the desired standard. The cement is the only thing that glues the particles together and makes the dam a watertight monolith and to save to the limit on cement is not good economy.

When calculating stresses in accordance with assumptions shown in Fig. 1 on work of average importance a maximum compression of 600 lb. per sq. in. and a maximum tension of 100 lb. per sq. in. are generally used. Maximum tension should not be paired with maximum compression, as in case the dam should crack in the tension side and thereby relieve the tension, more compression may be thrown on the compression side, although not always.

Sometimes a crack in the tension side (relieving the tension) may also relieve the compression on the compression side, depending upon whether the bending moment is created by a large force acting through a small eccentricity or by a small force acting through a large eccentricity. Whenever a large compression exists at the downstream face at the abutments it is safer not to allow a large tension at the upstream side of the abutments. If such a combination should happen, the thickness of the arch ought to be increased. The abutment stresses are the controlling stresses, as here the bending moment is the greatest.

If the simple arch formula were used in order to obtain a preliminary section using between 300 and 350 lbs. per sq. in. for the arch compression, it will generally be found that the section will be ample at the crown and that additions to the thickness only need be made at the abutments in order to satisfy the Cain formulas using the extreme stresses up to 600 lb. compression and 100 lb. tension per sq. in.

Should the assumption shown in Fig. 3 be used, it would be safe to allow somewhat higher unit stresses, since the condition represented by it is more severe than actually is the case.

Even using the assumptions shown in Figures 1 and 2 will indicate higher stresses than actually exist, for several reasons:

1. The cantilever action perhaps takes some of the load.

2. The concrete especially near the upstream face is wet and expands, relieving the tension.

3. In case the stresses are very high (higher than 800 lb. per sq. in. compression) a redistribution may take place due to the action of the time factor in creating "flow."

4. The arch may not be fixed in an extreme sense, hence taken all in all, the assumed designing stresses (600 lb. per sq. in. compression and 100 lb. per sq. in. tension) will not materialize at the abutments where the stresses generally are the highest.

A factor of safety of between 4 and 5 for the structure should therefore be a reality.

During the last 16 years, 40 or more Constant Angle Arch Dams have been built, besides many ordinary arch dams. Several large Constant Angle Arch Dams are being constructed at present, and this type of dam can therefore hardly be called

new and experimental any longer. The Constant Angle Arch design has been previously explained* and a detailed description of two large arch dams—the Lake Spaulding Dam, (California) and the Salmon Lake Dam, (Alaska) given.

Since then improvements in the design of arch dams have been effected partly due to practical experience gained in building numerous dams and partly due to improvements in the theory as recently advanced by different writers on the subject.

In the following the description of several typical Constant Angle Arch Dams recently built will be given indicating the successive improvement in design.



FIG. 4A—BULLARDS BAR DAM UNDER CONSTRUCTION. HEIGHT 183 FT. ABOVE FOUNDATION; SPAN 440 FT.

Figures 4, 5, 5A and 5B show working details of the Bullards Bar Dam built on the North Fork of the Yuba River, California. This dam is 182 ft. high above the foundation (175 ft. above river level) and creates a head of 175 ft. which is utilized in a power

*Trans. Am. Soc. C. E., 1915, v. 78, p. 685.

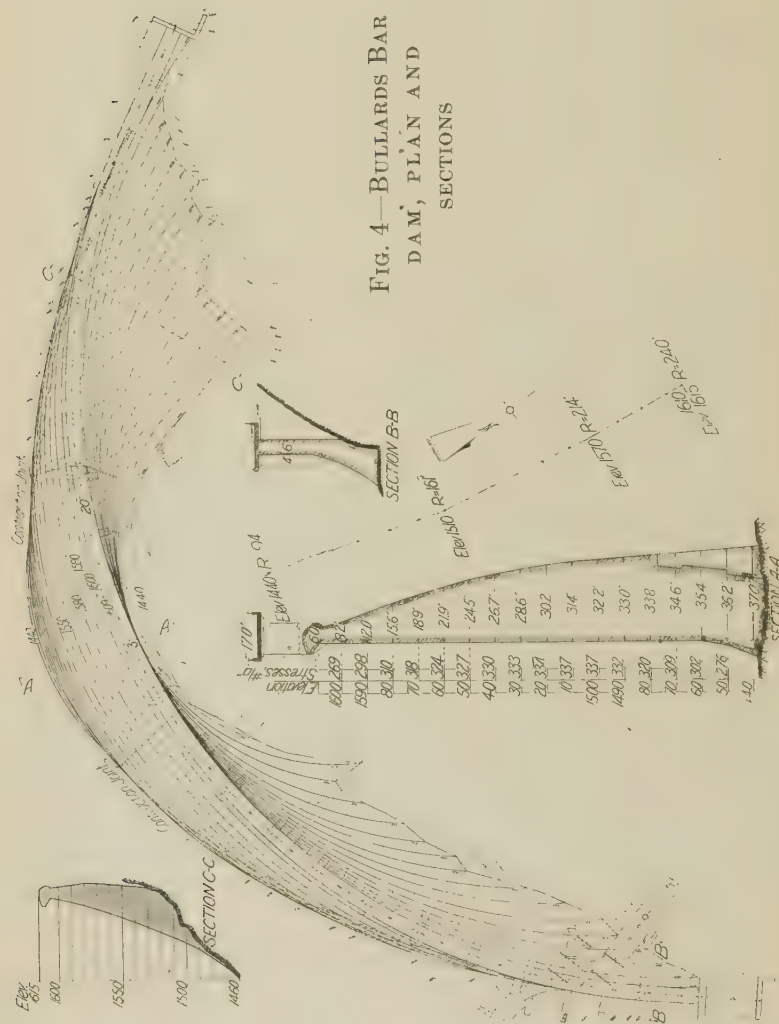


FIG. 4—BULLARDS BAR DAM, PLAN AND SECTIONS

plant located 300 ft. below the dam. The length of the upstream radius varies from 240 ft. at the crest to 94 ft. at the bottom. The development is now operated as part of the Pacific Gas & Electric Company's system. The reservoir space created by this dam is principally to be used for storing debris from gold mines located on the drainage above. Hydraulic gold mining was discontinued on a large scale in the early eighties when dumping of gravel or debris into any tributary to the Sacramento or San Joaquin Rivers was prohibited because these rivers were being rapidly filled up with gravel from the mines. With reservoirs built on the tributaries for storing this gravel, hydraulic mining is again legally possible.

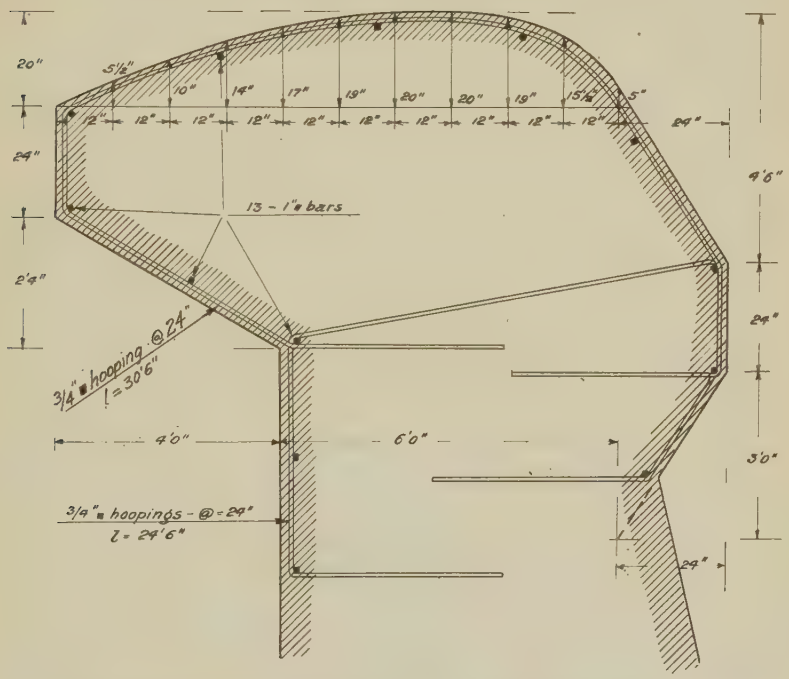


FIG. 5 —SECTION THROUGH SPILLWAY LIP, BULLARDS BAR PROJECT

The Bullards Bar reservoir will store approximately 70 million cu. yds. of debris to within 15 ft. of its crest. This represents also the approximate yardage of gold bearing gravel still remaining to be worked by the hydraulic method above this dam.

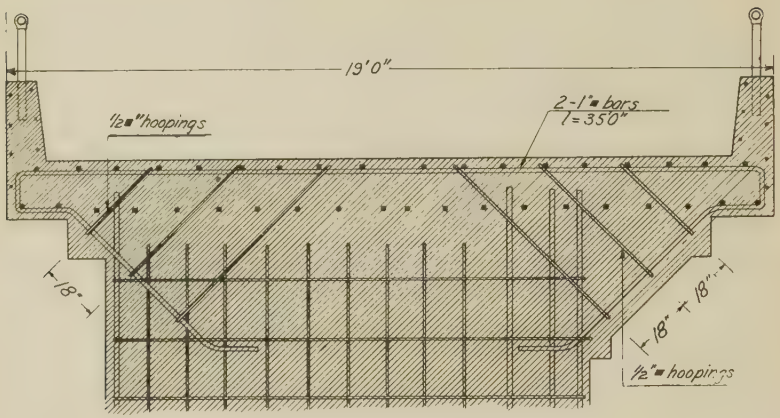


FIG. 5A—SECTION THROUGH ROADWAY, BULLARDS BAR PROJECT

Filling the reservoir with gravel and sand will not seriously interfere with the development of power, and since this dam is designed for a 13 ft. overflow it can be provided with 13-ft. gates along the crest whereby the fluctuating water storage will be increased by 8,000 acre ft.

This dam is designed to pass 65,000 sec. ft. over the crest. The normal winter flow, which is much less, requires only the three middle openings and it is intended to keep the tainter gates along the crest closed except a few in the middle in order to keep the head up to a maximum.

The dam has been in operation four winters without tainter gates and considerable experience has been obtained from watching the action of the overflow. The overflow period has been about 6 months every year, and the greatest depth of water going over the crest has so far been 11 ft. equivalent to 50,000 sec. ft.

In the middle of the dam the sheet of falling water stays clear of the dam and falls on a loose rock fill, if considerable water is flowing a pool is formed automatically.

On the sides however, the water first leaves the crest and then later strikes the concrete lower down under a small angle and finally strikes the rough rock abutment which is very solid.

In the middle no wear and tear can be detected. On the sides there is some wear of the concrete where the water strikes the

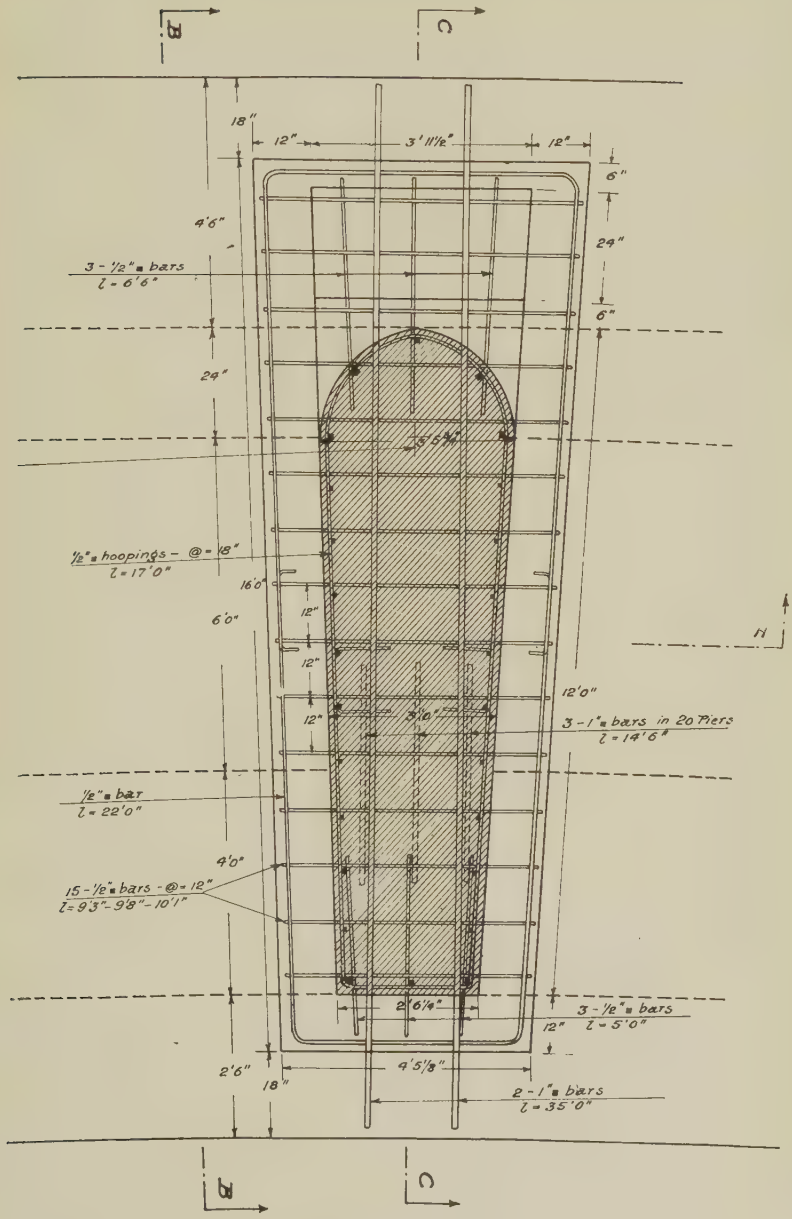


FIG. 5B—PLAN OF PIER, BULLARDS BAR PROJECT

dam body under a small angle. The wear on the rock is considerable however, and in order to prevent further wear tainter gates are now being installed on the sides to prevent the spill from going over the crest at that point and to confine it to the middle portion of the dam where no wear takes place.

In a river which is fairly level the writer has found that a mat of loose rocks (from the excavation) is a very safe bed for the overflow to fall on. The sheet of water striking the fill will split into many little jets whereby the energy will quickly be destroyed. The mat should be about 15 ft. thick under a fall of over 150 ft. During the first overflow period the mat will become two to three feet thinner where the water strikes it due to the pounding it receives, and after that nothing except dynamite can remove the rock.

On the Bullards Bar project a small dam for the purpose of forming a pool and for holding the rock fill back was provided. The secondary dam was about 5 ft. high and was completely buried in the rockfill. The rockfill stayed in place below the dam just as well as above, showing a secondary dam was unnecessary, and none has been provided on 14 free, overflow arch dams with which the writer has been connected.

For calculating the stresses in the dam, the assumption has been made that the reservoir was full of debris to within 15 ft. from the crest with 28 ft. of water on top of the debris. The weight of the debris was taken to equal 90 lb. per cu. ft. A reinforced concrete bridge along the crest of the dam takes the main county road across the canyon. This bridge has been designed for two 15-ton trucks meeting each other on the 20 ft. spans. A small forebay on the west side keeps the debris away from the pressure pipe intake.

The contraction joints are provided with a grout pipe system.

This dam was let by contract to the lowest bidder, Bent Bros., Inc., Los Angeles, and the following are the unit prices paid them on the main items:

Loose dirt excavation—2,500 cu. yd.	@	$ 2.50
Loose rock excavation—2,000 cu. yd.	@	2.50
Solid rock excavation—1,000 cu. yd.	@	5.00
Plain 1:3:6 concrete not including the cement—46,000 cu. yd.	@	6.45

Reinforced concrete 1:2:4 not including the cement—1,800 cu. yd.	@	$19.00
Installation of grout pipes	per ft.	1.00
Cement grouting—50 sacks	@	2.00

The dam is located 40 miles from the nearest railroad station at Marysville, California.

Very good concrete material in the form of river gravel was found in sufficient quantity 1¼ miles from the dam. This gravel was screened and remixed in the proper proportion for maximum density, using 2½-in. rock as the maximum size for best grading. Rocks up to 6 in. were left in the mix, but there were too few for complete grading. The actual mix was, most of the time, 1:2.8:6.5. The mixing plant was located high up on the hillside and the concrete was deposited by gravity through a chuting system of the Insley type. The sand (1:3 mortar) tested 12 per cent better than standard Ottawa sand in tension and 25 per cent better in compression at all ages.

Cylinders of concrete, 8 in. diameter and 16 in. long, were tested regularly, as the construction progressed, at the age of about 30 days. A test result (the average of six cylinders) picked out at random reads: 1673 lb. per sq. in. when 28 days old (Sept. 11th, 1923). The specimens were taken from the concrete in the dam. When fully aged this concrete can be expected to develop a crushing strength around 3000 lb. per sq. in. The weight per cu. ft. was 150 lb. The design of this dam, was made by A. C. Hoff, chief designing engineer for the Constant Angle Arch Dam Co. The testing was in charge of Theodore P. Dresser, Jr., chief engineer of Abbot A. Hanks, Inc., San Francisco, who also was in charge of testing on several of the dams mentioned later. George Ray was resident engineer and B. T. Millard his assistant engineer.

The constant angle arch dam which will pass a great amount of overflow over its crest is the Kerckhoff dam on the San Joaquin River, California, described by B. F. Jakobsen.* The tainter gates are 14 ft high. With a water level 14 ft. above the crest 60,000 sec. ft. will flow over the crest, and 100,000 sec. ft. can go over without interfering with the bridge on top. This dam has been in use for the last ten years. The overflow takes place about four to five months every year. It falls upon a loose rockfill made from the excavation and there is no secondary

*Trans. Am. Soc. C. E., 1921, Vol. 84, p. 106-107.

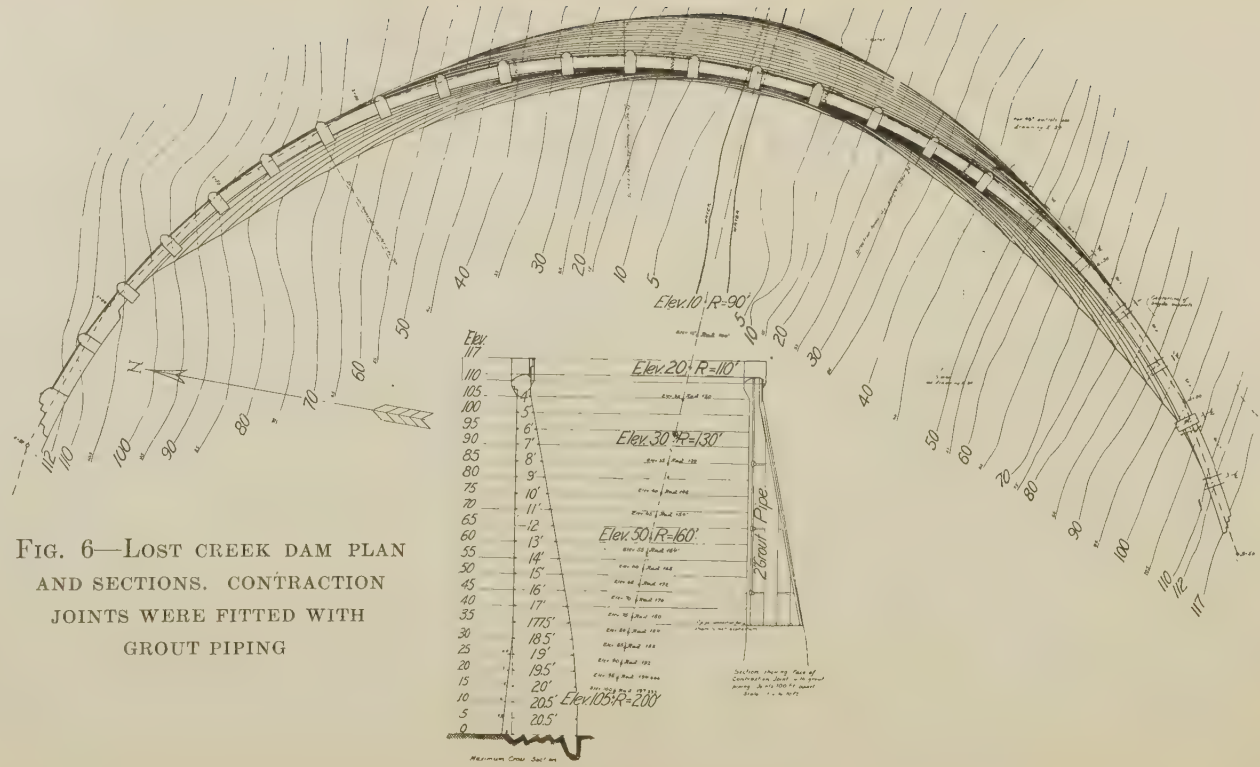


FIG. 6—LOST CREEK DAM PLAN AND SECTIONS. CONTRACTION JOINTS WERE FITTED WITH GROUT PIPING

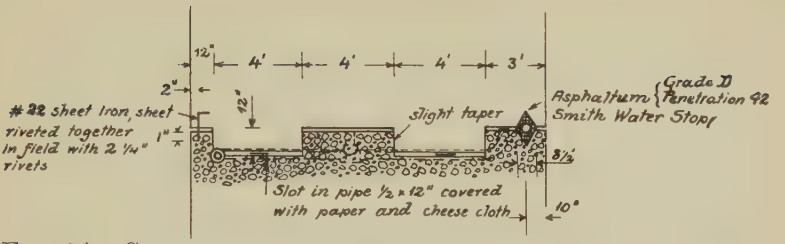


FIG. 6A—SECTION LOST CREEK DAM AT ELEVATION 45 SHOWING GROUT PIPES AND WATER STOP

dam. The Aluminum Co. of America has just fiinished a Constant Angle Arch Dam on the Little Tennessee River 180 ft. high, designed for 200,000 sec. ft. of overflow.

In Figures 6 and 6A, an arch dam 112 ft. high is shown in detail, having a variation in the length of its upstream radius of from 200 ft. at the top to 90 ft. at the bottom. It stores water for the Oroville-Wyandotte Irrigation District, of which S. J. Norris was chief engineer. F. E. Baxter was resident engineer.

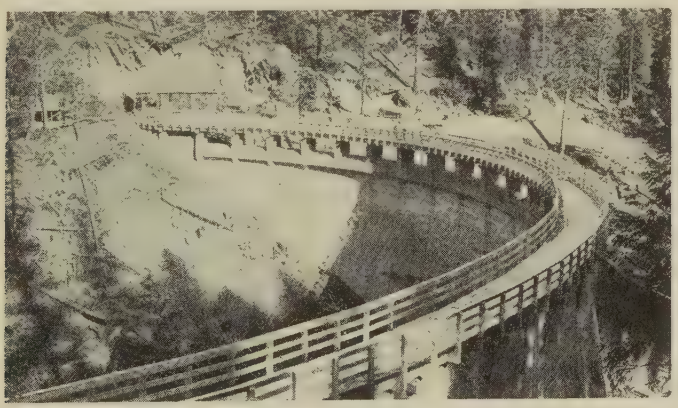


FIG. 6B—LOST CREEK DAM, 112 FT. HIGH, 380 FT. SPAN. THICKNESS AT CREST 4 FT.; AT BOTTOM 235 FT.

It is seen that the canyon is well adapted to a Constant Angle Arch type of dam, being of a quite regular "V" shape. All the centers lie on a straight line which makes it easy to lay out the dam. A wooden bridge on piers along the crest accommodates an unimportant county road. The contraction joints are provided with grout pipes and Smith patented waterstop.

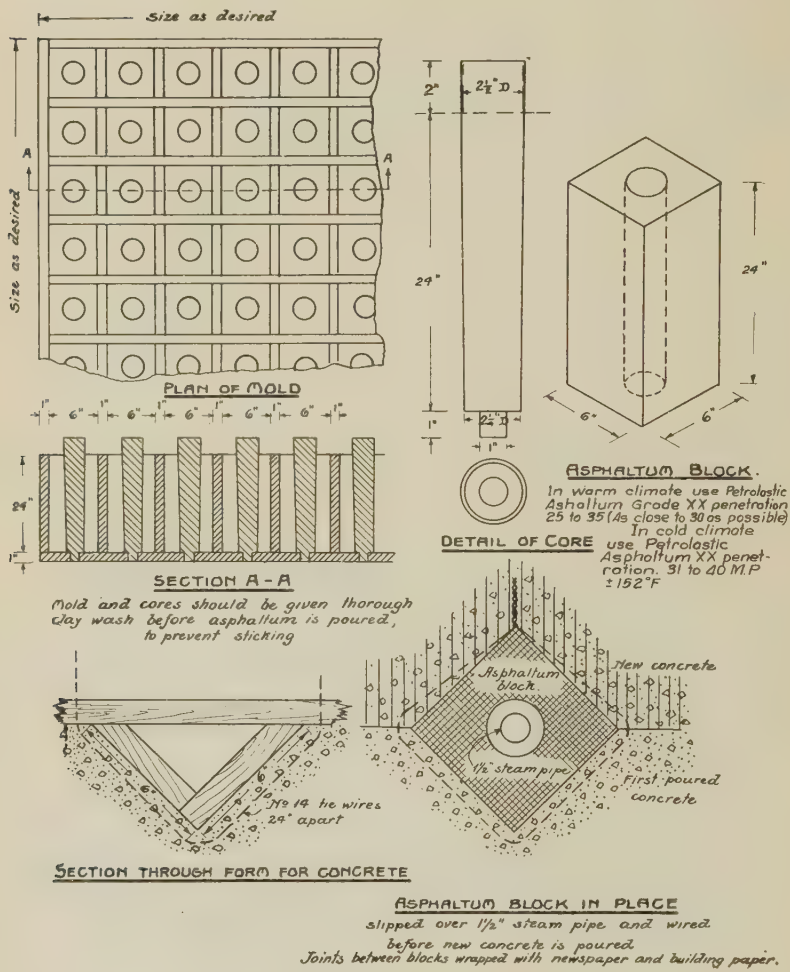


FIG. 7—DETAILS OF SMITH WATER STOP

A detail of the Smith waterstop is shown on Fig. 7. It consists of precast asphaltum blocks strung on an iron pipe and built in across the contraction joints close to the upstream face. The asphaltum may also be poured in around the pipe in say, 5 ft. lifts. Later steam is let through the pipe and the blocks melt

together and fill the diamond shaped hole and any cavity which may exist. This waterstop was first brought out by Francis B. Smith, member, American Society Civil Engineers.

The dam is an overflow dam provided with flashboards 3 ft. high. The normal overflow is confined by means of these flash boards to the middle portion where it falls clear of the dam. With 5 ft. of water going over the entire crest the discharge is 15,000 sec. ft. The dam was built by contract, Messrs. Lord and Bishop of Napa, California, being low bidders and being awarded the contract at the following unit prices on the main items:

570 cu. yd. of loose rock and gravel excavation	@	$ 1.25
2378 cu. yd. of solid rock excavation	@	5.00
11,093 cu. yd. of 1:3:6 concrete, including everything except the cement	@	5.15

The dam is located in rough country, 50 miles from the nearest railroad. The concrete material was found within a mile of the dam and consisted of gravel, which was screened and remixed in the proper proportion. When the work was half completed the

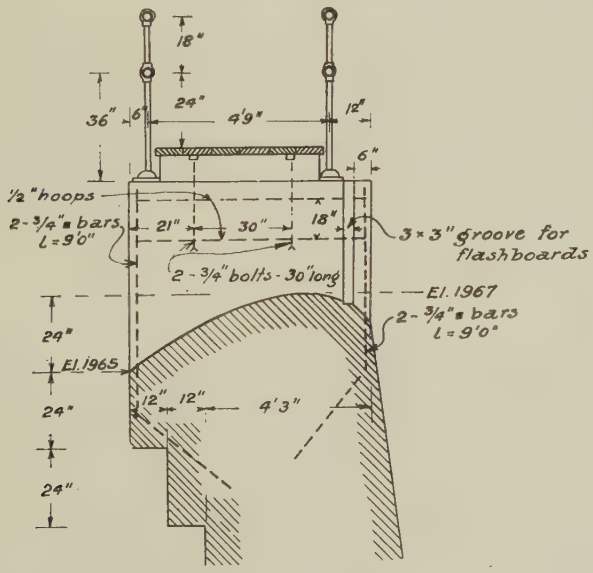


FIG. 8A—CONCOW DAM SECTION OF SPILLWAY AND BRIDGE

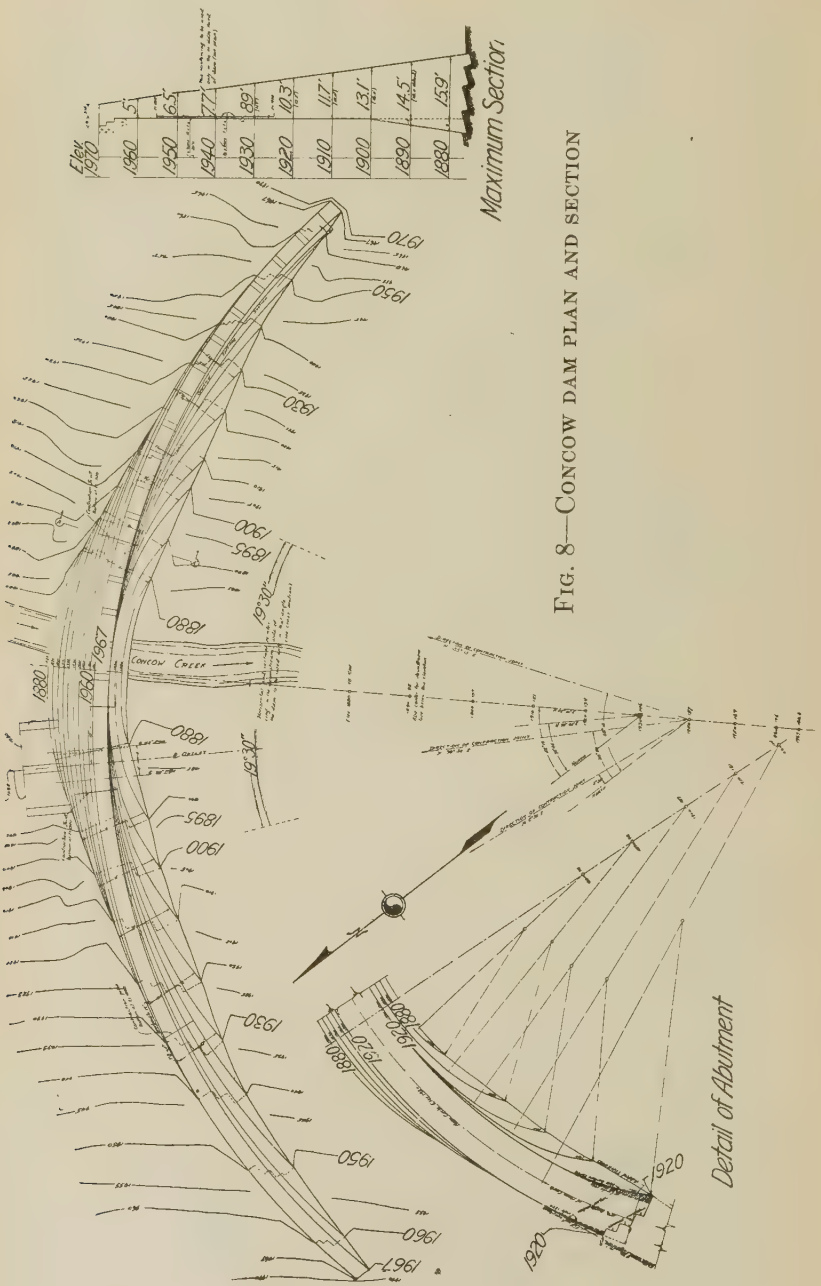


FIG. 8—CONCOW DAM PLAN AND SECTION

deposit became deficient in coarse material. A quarry was then opened up and crushed rock was mixed with the gravel.

The larger sand particles were slightly weak, but the average quality of the sand particles was satisfactory. The sand (1:3 mortar) gave low seven-day strengths (tensile 82 per cent and compression 68 per cent of standard Ottawa sand), but had high 7-day to 28-day gains, giving 28-day tensile and compressive strength 100 per cent and 92.5 per cent respectively of standard Ottawa sand.

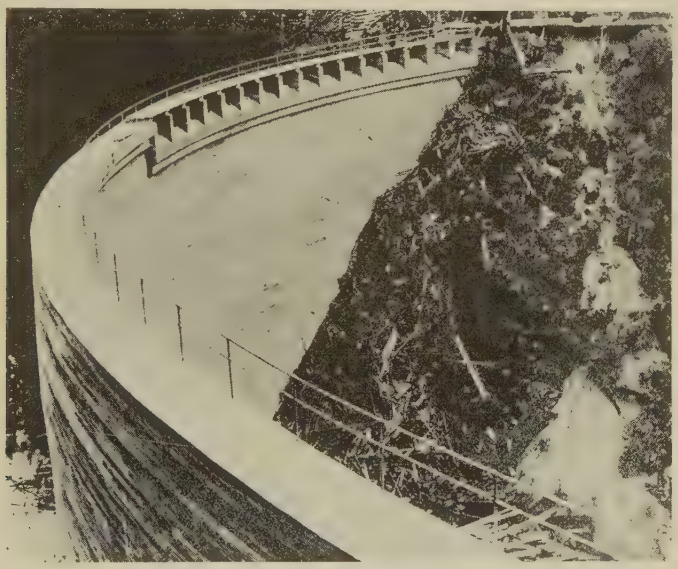


FIG. 8B—CONCOW DAM, 90 FT. HIGH ABOVE STREAM BED. SPAN 275 FT.

Somewhat over 1 barrel (average addition 15 per cent) of cement per cubic yard was necessary to bring the crushing strength of the concrete up to 1500 lb. per sq. in. when 28 days old. Ultimately, this concrete should develop a strength of at least 2500 lb. per sq. in. The concrete was graded to a maximum size of 3 in.; it was mixed for 1¼ minutes after all the ingredients were deposited in the mixer and distributed through a chuting system of the Insley type. This design and also the design of the Bullards Bar dam was made using the simple arch formula.

TABLE TO ACCOMPANY FIG. 9

Numbers refer to formulas as given by Wm. Cain in paper No. 1483, Transact. A.S.C.E., 1922.		Dam—Crest Elev. 1967					Dam—Crest Elev. 1947		
		Elev. 1890	Elev. 1900	Elev. 1910	Elev. 1920	Elev. 1930	Elev. 1890	Elev. 1900	Elev. 1910
	t_1	16.0′	13.1′	11.7′	10.3′	8.9′	12.5′	9.6′	8.2′
	t_2	18.0′	16.0′	15.0′	13.5′	11.7′	13.0′	10.5′	9.5′
	R_u	92′	107′	121′	134	146′	92′	107′	121′
	$r = R_u - \frac{t_1}{2}$	84.0′	100.1′	115.15′	128.85	141.55′	85.75′	102.2′	116.9′
	$2\phi^\circ$	64°	68°	73°	79°	83°	64°	68°	73°
	ϕ°	32°	34°	36.5°	39.5°	41.5°	32°	34°	36.5°
	$\phi = \phi^\circ \times \frac{\pi}{180} = \phi \times 0.01745$	0.55840	0.59330	0.63693	0.68928	0.72418	0.55840	0.59330	0.63693
	$\sin 2\phi$	0.89879	0.92718	0.95630	0.98163	0.99255	0.89879	0 92718	0.95630
	$\sin \phi$	0.52992	0.55919	0.59482	0.63608	0.66262	0.52992	0.55919	0.59482
	$\cos \phi$	0.84805	0.82904	0.80386	0.77162	0.74896	0.84805	0.82904	0.80386
	$\frac{\sin \phi}{\phi}$	0.94900	0.94251	0.93389	0.92283	0.91500	0.94900	0.94251	0.93389
	$K^2 = \frac{1}{12} \times t_1^2$	21.33	14.3	11.41	8.84	6.60	13.02	7.68	5.60
11	$D = \left(1 + \frac{K^2}{r^2}\right) \times \phi \times (\phi + \frac{1}{2} \sin 2\phi) - 2 \sin 2\phi$	0.0028241	0.002537	0.0032086	0,0046441	0.0059800	0.0021218	0.002128	0.0028885
	$D_s = D + 2.88 \times \frac{K}{r^2} \times \phi \times (\phi - \frac{1}{2} \sin 2\phi)$	0.0033540	0.002851	0.0034592	0.0048539	0.0062016	0.0024322	0.002291	0.0030079
	$\frac{D}{D_s}$	0.8420	0.8900	0.92755	0.9568	0.9643	0.8724	0.9289	0.9603
	$\frac{2 \sin \phi}{D}$	375.28	440.8	370.77	273.93	221.61	499.50	525.60	411.85
	P	4813	4188	3563	2938	2313	3563	2938	2313
	$P_1 = P \times \frac{R_u}{r}$	5271	4460	3744	3055	2386	3823	3076	2394
	$P_1 \times r$	442,800	447,800	431,100	393,600	337,700	327,800	314,400	279,900
10	$p_1 r - P_o = \frac{2 \sin \phi}{D} \times p_1 \times \phi \times \frac{K^2}{r}$	280,500	166,100	87,600	46,400	17,800	161,900	72,100	30,100
	P_o	162,300	281,700	343,500	347,200	319,900	165,900	242,300	249,800
12	$M_o = \therefore (p_1 r - P_o) \times r - \left(\frac{\sin \phi}{\phi}\right)$	1,201,700	958,700	666,900	461,400	214,200	708,000	423,600	232,600
	$M_o^1 = M_o \times \frac{D}{D_s}$	1,011,800	853,200	618,600	441,500	206,600	617,700	393,500	223,400
4	$P_1 = p_1 r - (p_1 r - P_o) \cos \phi$	204,900	310,100	360,700	357,800	324,400	190,500	254,600	255,700
13	$M_1 = (p r - P_o) \times r \left(\frac{\sin \phi}{\phi} - \cos \phi\right)$	2,378,600	1,892,000	1,311,600	904,000	418,400	1,398,700	836,100	457,500

	2,378,600	1,684,000	1,216,600	864,900	403,500	1,220,200	776,700	433,300
Extr.	+234	+356	+392	+407	+358	+257	+353	+349
Intr.	−94	−58	+16	+61	+140	−71	−3	+73
Intr.	+336	+409	+392	+382	+317	+403	+462	+390
Extr.	−178	−139	−58	−14	+71	−199	−126	−16

$$M'_1 = M_1 \times \frac{D}{D_s}$$

Abutm. Crown

$$S_0 = \frac{P_0}{t_1 \times 144} \pm \frac{6\,M'_0}{t_1^2 \times 144}$$

$$S_1 = \frac{P_1}{t_2 \times 144} \pm \frac{6 \times M'_1}{t_2^2 \times 144}$$

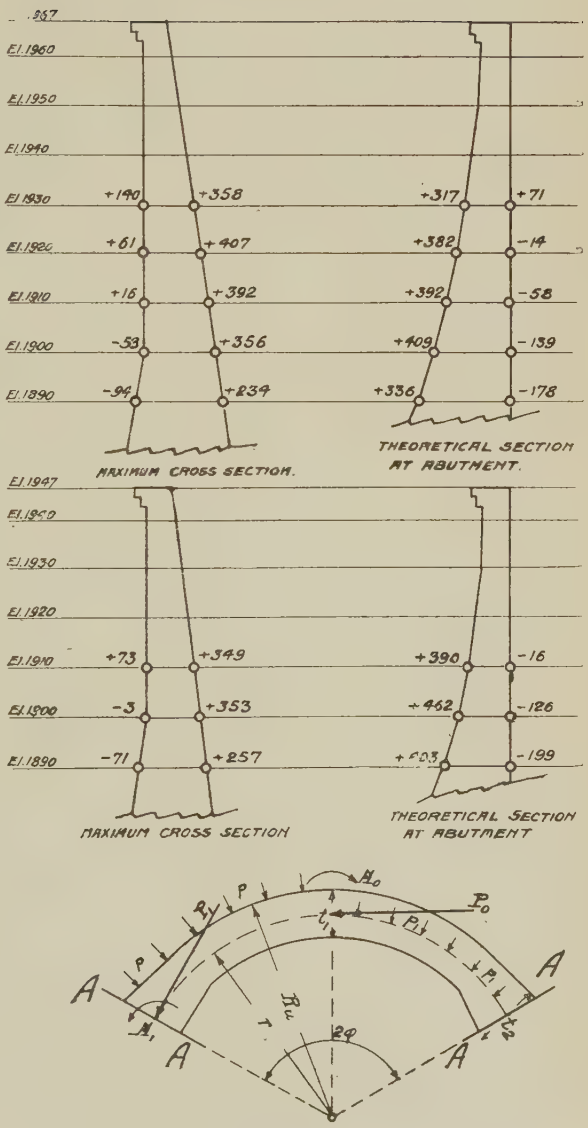


FIG. 9—DIAGRAM FOR CALCULATION OF CONCOW DAM

CONCOW CREEK DAM

In Fig. 8 details are shown on the Concow Creek dam, also called Lake Willenor dam, built for the Thermalito and Table Mountain Irrigation Districts, of which S. J. Norris was chief engineer and F. E. Baxter was resident engineer. The Dam is about 100 ft. high, with length of upstream radius at crest 180.6 ft. and at bottom 78 ft.

In the design of this dam, the method in Fig. 2 was used. When applying the Cain formulas p was taken as the full water pressure. The calculations are shown on Fig. 9 and the accompanying table. It is seen that the tension at the abutments is rather high, but since the method in Fig. 2, plane AA has been used in this particular case, this tension represents the imaginary tension inside the rock.

The dam is thicker at the abutments than further out on the arch to bring the stresses down to a conservative value. This additional thickness is made decreasing from a maximum amount at the rock junction to zero approximately 15 per cent out from the abutments. In this way the material is put where it does the most good. This dam is also an overflow dam, provided with flashboards 3 ft. high so as to keep water level to the maximum elevation possible after the rainy season has passed and to direct the overflow towards the middle.

The dam was let by contract to the lowest bidder, J. S. Greaves, Oroville, California, at the following prices:

905 cu. yd. of loose rock	@	$ 1.50
2,156 cu. yd. of solid rock	@	7.90
6,332 cu. yd. of 1:2½:5 concrete, including everything except cement	@	7.45
Per cu. yd. of reinforced 1:2:4 concrete, exclusive of cement and steel	@	19.50
Per ft. of grout pipe installed (exclusive of pipe)	@	.25

The dam is located 26 miles from the nearest railroad station (Oroville, California). Being a slim structure, a richer concrete mix than usual was used with 1¼ barrels of cement per cu. yd. of concrete. The weight of the concrete was 149 lb. per cu. ft. and the strength at 28 days averaged close to 1900 lb. per sq. in., fluctuating with the variation in the amount of cement used and the season in which the specimen was made, whether in the hot or in the cold. It was not possible to keep the cylinders (6 in. x 9 in. and 10 in. x 15 in.) under constant temperature in the

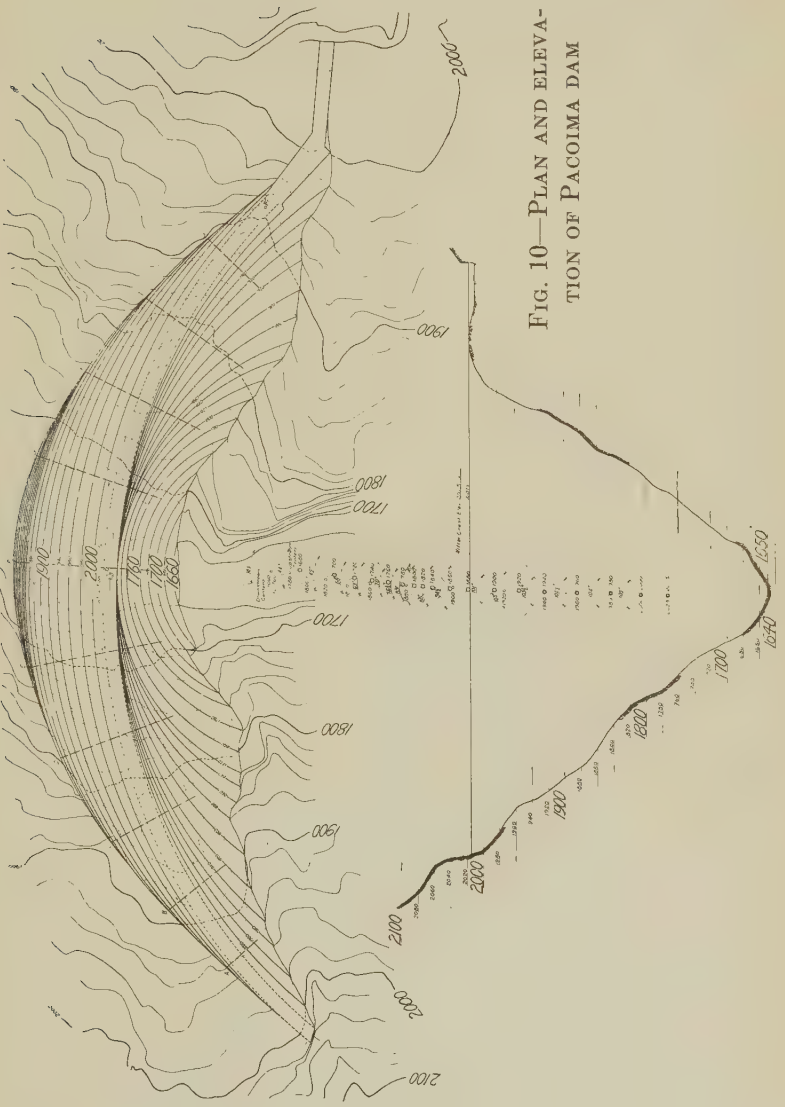


FIG. 10—PLAN AND ELEVATION OF PACOIMA DAM

field. The mixing plant was located at a sufficient elevation above the dam to allow chuting the concrete in place. Sand and gravel of good quality were found very close to the dam site.

PACOIMA DAM

In Figures 10, 10A and 10B, details are shown of the Pacoima Dam, built by the Los Angeles County Flood Control District of which J. W. Reagan was chief engineer until three years

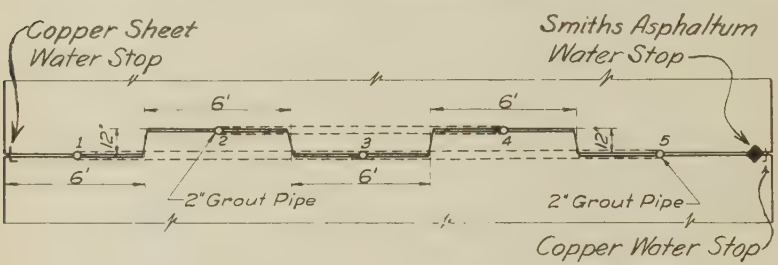


FIG. 10B—PACOIMA DAM. TYPICAL PLAN OF CONTRACTION JOINT

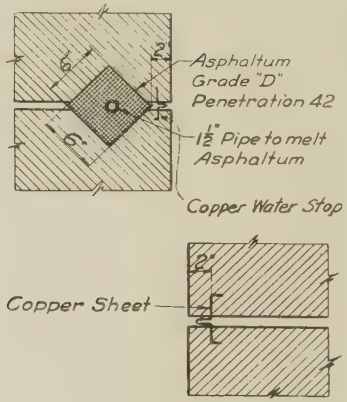


FIG. 10B—DETAIL OF SMITH'S ASPHALTUM WATER STOP, PACOIMA DAM

FIG. 10B—DETAIL OF COPPER SHEET WATER STOP

ago, when he became consulting engineer and E. Court Eaton became chief engineer. As the name implies, the purpose of the reservoir formed by the dam is to take care of flood waters due to torrential rains. The total storage in this reservoir amounts to 12,000 acre feet only, but it is intended to utilize this storage capacity several times per season by alternately emptying and

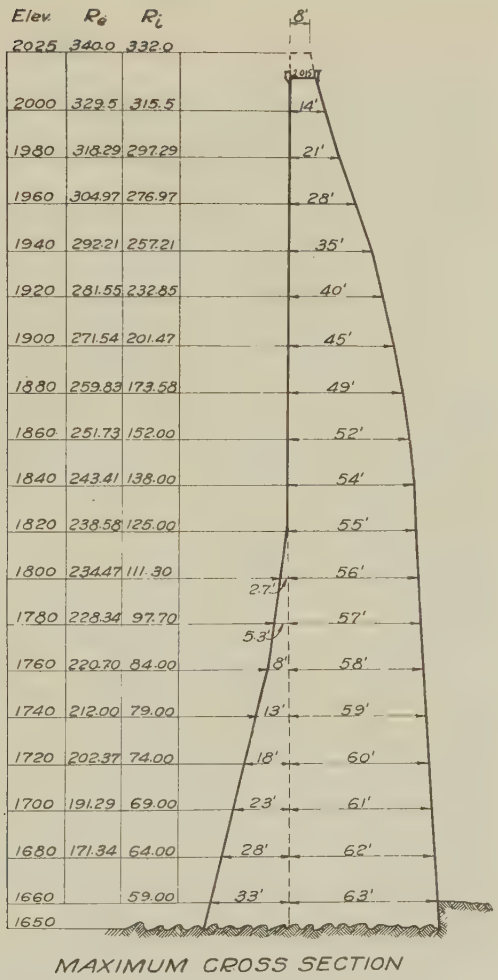


FIG. 10A—PACOIMA DAM. MAXIMUM CROSS SECTION

filling it as many times as the particular season may require. The water from the reservoir is to be let out at a rate slow enough to avoid damage to land below, and if possible to be all absorbed by the so-called gravel cone at the head of the valley, to help supply the irrigation pumping demand during the dry season.

The construction work was let to the lowest bidder, Bent



FIG. 10C'—PACOIMA DAM. HEIGHT ABOVE FOUNDATION 373 FT.; CREST LENGTH 680 FT.; 225,000 CU. YDS. PLAIN CONCRETE

Brothers, Inc., Los Angeles, at the following unit prices on the most important items:

95,000 cu. yd. of mixed excavation	@	$ 4.00
2,000 cu. yd. of rock excavation, no blasting	@	6.00
225,000 cu. yd. of plain concrete, exclusive of cement	@	4.70
620 cu. yd. of reinforced concrete, exclusive of steel and cement	@	11.00
1,200 lin. ft. of 2 in. galvanized iron pipe railing to place	@	.40
1,900 lin. ft. of asphaltum water stop with steampipe—to place	@	.46
1,000 lin. ft., more or less, 3 in. diameter holes into bedrock, placing pipe and pressure grouting under 175 lb. per sq. in. pressure	@	4.00

The concrete material, gravel, was available within a mile of the damsite and the railroad (San Fernando station of the Southern Pacific) is only about four miles away. The crest length is 700 ft. On account of the great height of the dam, 373 ft. above the foundation and 365 ft. above streambed, and the importance of the location 20 miles above the City of Los Angeles, it was decided to use five sacks of cement per cubic yard of concrete, although the sand available is of an excellent quality and grading, testing at least 25 per cent higher than standard Ottawa sand.

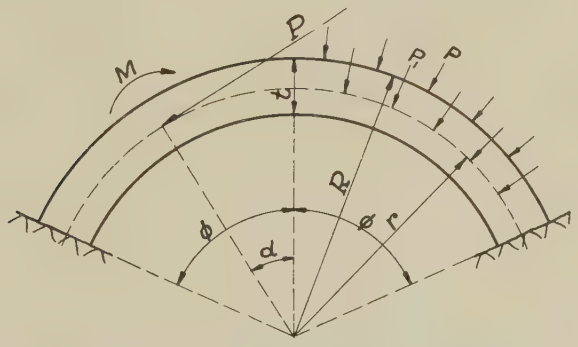


FIG. 11—DERIVATION OF CONSTANTS FOR DESIGN OF ARCH DAMS

TABLE TO ACCOMPANY FIG. 11

EQUATIONS 1 TO 7 INCLUSIVE ARE FROM CAIN'S ANALYSIS

	Number of Formula in Cain's Paper.
① $P = p_1 r - (p_1 r - P_o) \cos \alpha$	④
② $M = (p_1 r - P_o)\, r \left(\frac{\sin \phi}{\phi} - \cos \alpha\right)$	⑬
③ Where $p_1 r = p R$	page 235
④ $p_1 r - P_o = 2 \sin \phi \, \frac{\phi}{D_1} \, \frac{k^2}{r} \, p_1$	⑲

⑤ $D_s = D + 2.88\frac{k^2}{r^2}\,\phi\,(\phi - \tfrac{1}{2}\sin 2\,\phi)$ page 269

⑥ $D = \left(1 + \frac{k^2}{r^2}\right)\phi\,(\phi + \tfrac{1}{2}\sin 2\,\phi) - 2\sin^2\phi$ ⑪

⑦ $k^2 = \frac{1}{12}t^2$ page 236

⑧ Let $A = \left(\frac{r}{t}\right)^2$ then $\left(\frac{k}{r}\right)^2 = \frac{1}{12A}$ ⑨

④ $p_1 r - P_o = 2\sin\phi \times \frac{\phi}{D_s} \times \frac{k^2}{r} \times p_1$ but $\frac{k^2}{r} = \frac{r}{12A}$ and from ③

⑩ We have $p_1 r - P_o = \frac{\sin\phi \times p.R \times \phi}{6AD_s}$

from ⑤ & ⑥ we have

$$D_s = \left[\left(1 + \frac{1}{12A}\right)\phi\,(\phi + \tfrac{1}{2}\sin 2\,\phi) - 2\sin^2\phi\right] + \left[\frac{2.88}{12A}\,\phi\,(\phi - \tfrac{1}{2}\sin 2\,\phi)\right]$$

$$\therefore 6AD_s = \phi\left[6A\left(\phi + \tfrac{1}{2}\sin 2\,\phi - \frac{2\sin^2\phi}{\phi}\right) + \tfrac{1}{2}\,(\phi + \tfrac{1}{2}\sin 2\,\phi) + 1.44\,(\phi - \tfrac{1}{2}\sin 2\,\phi)\right]$$

⑪ $6AD_s = \phi\left[A\left(6\,\phi + 3\sin 2\,\phi - 12\frac{\sin^2\phi}{\phi}\right) + 1.94\,\phi - .47\sin 2\,\phi\right]$

⑫ let $K_1 = 6\,\phi + 3\sin 2\,\phi - 12\frac{\sin^2\phi}{\phi}$

⑬ and $K_2 = 1.94\,\phi - .47\sin 2\phi$
then $6AD_s = \phi\,[AK_1 + K_2]$

$\therefore$ ⑭ $p_1 r - P_o = p\,R - P_o = p\,R\,\frac{\phi\sin\phi}{\phi\,(AK_1 + K_2)}$

from ① & ③ $P = p\,R - (p\,R - P_o)\cos\alpha = p\,R\,\frac{(AK_1 + K_2) - \sin\phi\cos\alpha}{AK_1 + K_2}$

$\therefore$ ⑮ $P = p\,R\,\frac{AK_1 + K_3}{AK_1 + K_2}$

⑯ Where $K_3 = 1.94\,\phi - 0.47\sin 2\,\phi - \sin\phi\cos\alpha$

from ② $M = (p\,R - P_o) \times r \times \left(\frac{\sin\phi}{\phi} - \cos\alpha\right)$

$= p\,R\,\frac{\phi\sin\phi}{\phi\,(AK_1 + K_2)} \times r \times \left(\frac{\sin\phi}{\phi} - \cos\alpha\right)$

⑰ $= p\,R\,r\,\frac{\frac{\sin^2\phi}{\phi} - \sin\phi\cos\alpha}{AK_1 + K_2} = p\,R\,r\,\frac{K_4}{AK_1 + K_2}$

⑱ Where $K_4 = \frac{\sin^2\phi}{\phi} - \sin\phi\cos\alpha$

let $C_1 = \frac{K_3}{K_1}$, $C_2 = \frac{K_4}{K_1}$, $C_3 = \frac{K_2}{K_1}$

⑲ then $P = p \times R \times \frac{A + C_1}{A + C_3}$

⑳ and $M = p \times R \times r \times \frac{C_2}{A + C_3}$

The values of C_1, C_2 & C_3 are as follows:

$$C_1 = \frac{K_3}{K_1} = \frac{1.94\,\phi - 0.47\sin 2\,\phi - \sin\phi\cos\alpha}{6\,\phi + 3\sin 2\,\phi - \frac{12\sin^2\phi}{\phi}}$$

$$C_2 = \frac{K_4}{K_1} = \frac{\frac{\sin^2\phi}{\phi} - \sin\phi\cos\alpha}{6\,\phi\,\tfrac{1}{2}\,3\sin 2\,\phi - \frac{12\sin^2\phi}{\phi}}$$

$$C_3 = \frac{K_2}{K_1} = \frac{1.94\,\phi - 0.47\sin 2\,\phi}{6\,\phi + 3\sin 2\,\phi - \frac{12\sin^2\phi}{\phi}}$$

Reference—Paper No. 1483 Trans. A. S. C. E. 1922. "The Circular Arch under normal Loads" by Prof. Wm. Cain.

It is seen from Fig. 11A that the stresses have been kept moderate. It will also be noted from Fig. 11A and the accompanying table that a short cut method compared with Fig. 9 has been used in the calculations. This was devised by A. C. Hoff. The derivation of the constants in Fig. 11A is shown on Fig. 11. The stress sheet (Fig. 11) was calculated with the intention of constructing the dam to elevation 2025, but construction was temporarily stopped at elevation 2015. In the lower zone where apparent tension exists, in case the full water-load is figured as borne by the arch, this same full water load can also be borne by the shear action along the rock contact, creating only moderate shear stresses. The load therefore, divides between the two actions and actual tension in the faces can not exist to any extent. The action of Poisson's ratio also tends to eliminate tension in the faces towards lower levels.

The fact that concrete expands when wet is a well known phenomena. Professors Campbell and White at the University of Michigan, who have carried out a series of careful experiments extending over several years, give the expansion of concrete as between 0.02 and 0.04 per cent of its length when dry.

For concrete having a modulus of elasticity of 2,000,000 and a coefficient of expansion due to being wetted of 0.0002, such expansion would equalize a stretch due to a tension of 400 lb. per sq. in.

Along the upstream face where the heaviest tension (apparent or real) exists, the concrete is water-soaked and will, therefore, have its complete expansion with reservoir full.

Along the down stream face the concrete is less water soaked and it will not have expanded as much as 0.02 per cent, but the effect of this expansion in neutralizing the tension is not so much needed along the downstream face as along the upstream face.

The maximum foundation pressure exists at the bottom, being about 430 lb. per sq. in. along the inclined downstream face, in the middle. With Poisson's ratio between 1/5 and 1/6 the resulting axial or horizontal compression will be about 75 lb. per sq. in., at elevation 1650 on the downstream side at least in the middle, thus being able to compensate for a considerable arch tension at the downstream face. At elevation 1800 where the arch

TABLE TO ACC

2000	1980	1960	1940	1920	1900	1880	1860
329^{5}	318^{29}	304^{97}	292^{21}	281^{56}	271^{54}	259^{83}	251^{73}
14	21	28	35	40	45	49	52
1,563	2,813	4,063	5,313	6,563	7,813	9,063	10,303
105	104	102½	103½	103	101	94½	88
322.5	307.79	290.97	274.71	261.55	249.04	235.33	225.73
530.64	214.82	107.99	61.61	42.76	30.63	23.07	18.84
515,009	895,350	1,239,093	1,552,512	1,847,813	2,121,542	2,354,839	2,596,091
166.090	275.580	360.539	426.491	483.295	528.349	554.164	586.016
3.4707	3.5437	3.6531	3.5802	3.6166	3.7625	4.3069	4.9798
−0.69671	−0.71208	−0.73513	−0.71976	−0.72744	−0.75817	−0.87302	−1.01579
8.6631	8.9408	9.3817	9.0860	9.2312	9.8316	12.2665	15.5909
534.1107	218.3637	111.6431	65.1902	46.3766	34.3925	27.3769	23.8198
539.3031	223.7608	117.3717	70.6960	51.9912	40.4616	35.3365	34.4309
.99037	.97588	.95119	.92212	.89201	.85000	.77475	.69181
510,049	873,754	1,178,613	1,431,602	1,648,268	1,803,311	1,824,412	1,796,002
−214,567	−876,985	−2,258,151	−4,342,129	−6,762,070	−9,900,211	−13,691,120	−17,288,807
5.5021	5.6211	5.7997	5.6806	5.7401	5.9782	6.8675	7.9731
1.33477	1.36544	1.41146	1.38078	1.39612	1.45747	1.68659	1.97148
8.6631	8.9408	9.3817	9.0860	9.2312	9.8316	12.2665	15.5909
536.1421	220.4411	113.7897	67.2906	48.5001	36.6082	29.9375	25.8131
539.3031	223.7608	117.3717	70.6960	51.9912	40.4616	35.3365	34.4309
.99414	.98516	.96948	.95183	.93285	.90176	.84721	.77875
511,991	882,033	1,201,276	1,477,127	1,723,732	1,919,486	1,995,043	2,021,706
411,071	1,681,653	4,335,682	8,329,895	12,977,923	18,784,547	26,449,916	33,554,709
+299	+372	+412	+432	+462	+482	+497	+506
+207	+206	+172	+136	+110	+74	+21	−26
0	0	0	0	1	0	0	0
0	0	0	0	3	9^{5}	14	19
14	21	28	35	44	54^{5}	63	71
+341	+451	+528	+576	+551	+509	+498	+475
+167	+133	+68	+10	−7	−19	−58	−79
0	0	0	0	1	0	0	0
0	0	0	0	3	12	17	24
14	21	28	35	44	57	66	76
+341	+451	+528	+576	+551	+475	+463	+427
+167	+133	+68	+10	−7	−7	−43	−57

tension is the greatest (on the downstream side 63), it is found that the action of Poisson's ratio, due to the weight of the dam above just about compensates for this tension. The action of Poisson's ratio is as positive as the action of the elasticity of the material and can be taken into full consideration when looking for means of relieving the tension. The lateral expansion begins as soon as the vertical compression begins. It is not like the action of the time factor, dependent upon a certain high value of

OMPANY FIG. 11A

1840	1820	1800	1780	1760	Elevation	
243.40	238.68	234.57	228.34	220.70	R	
54	55	58.7	62.8	66	t_o	
11,563	12,813	14,063	15,313	16,563	p	
87	83	76½	70	63	2ϕ	
216.41	211.08	205.12	197.19	187.70	$r = R - \frac{1}{2} t_o$	
16.06	14.73	12.21	10.02	8.09	$A = \left(\frac{r}{t_o}\right)^2$	
2,814,550	3,056.926	3,297,352	3,496,570	3,655,454	$p \times R$	lbs. ft.
609.097	645.256	676.353	689.489	686.129	$p \times R \times r \times 10^6$	
5.0996	5.6225	6.6198	7.9189	9.7968	C_1	Crown m = o
−1.04061	−1.14965	−1.35929	−1.63217	−2.02737	C_2	
16.1834	19.0205	25.1966	34.6587	50.3341	C_3	
21.1596	20.3525	18.8298	17.9389	17.8868	$A + C_1$	
32.2434	33.7505	37.4066	44.6787	58.4241	$A + C_3$	
.65625	.60302	.50338	.40151	.30615	$K = \frac{A + C_1}{A + C_3}$	
1,847,048	1,843,388	1,659,821	1,403,908	1,119,117	$P_o = K \times p \times R$ lbs.	
−19.657,742	−21,979,484	−24,577,477	−25,187,914	−23,809,307	$M_o = \frac{C_2}{A + C_3} \times p \times R \times r$	
8.1652	9.0113	10.6375	12.7548	15.8176	C_1	Abutment m = 1
2.02096	2.23903	2.65838	3.20373	3.99335	C_2	
16.1834	19.0205	25.1966	34.6587	50.3341	C_3	
24.2252	23.7413	22.8475	22.7748	23.9076	$A + C_1$	
32.2434	33.7505	37.4066	44.6787	58.4241	$A + C_3$	
.75132	.70344	.61082	.50975	.40921	$K = \frac{A + C_1}{A + C_3}$	
2,114,628	2,150,364	2,014,089	1,782,377	1,495,848	$P_1 = K \times p \times R$	
38,177,136	42,803,700	48,086,472	49,440,484	46,897,654	$M_1 = \frac{C_2}{A + C_3} \times p \times R \times r$	
+519	+536	+493	+426	+346	Extrados	Crown Stresses
−43	−70	−101	−114	−110	Intrados	
0	0	0	0	0	Upstream Addition	Abutment Stresses
26	30	33	38	35	Downstream Addition	
80	85	91.5	100.5	101	t_1	
+433	+423	+391	+328	+295	Intrados	
−65	−71	−85	−82	−89	Extrados	
0	0	0	0	0	Upstream Addition	Abutment Stresses
27.5	30	30	27	26	Downstream Addition	
81.5	85	88.7	89.2	92	t_2	
+419	+423	+413	+397	+344	Intrados	
−59	−71	−97	−119	−118	Extrados	

unit stress being first reached in the material.

Due to the narrowness of the canyon, shear action predominates toward the bottom and pure arch action exists only towards the top. In the lower zone shear and arch action will divide the load between them and inasmuch as each action is sufficient to carry the whole load, the dam should be very safe for this portion. The shear cannot be transmitted from the middle to the abutments through the thick arch (horizontal

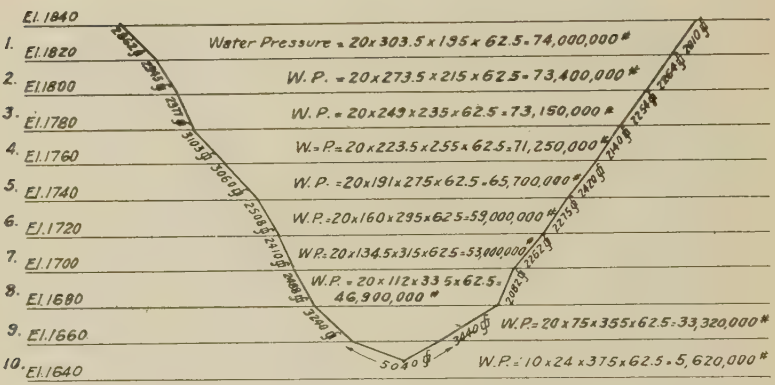


FIG. 11A—CALCULATION OF FINAL DESIGN, PACOIMA DAM

1. If all of the load *below El.* 1840 is assumed taken up by *Shear Action* the *Average Shear* in the martial along the area of rock contact will be

$$S = \frac{481,340,000 + 74,000,000}{144 \times (45708 + 2862 + 2010)} = \frac{555,340,000}{144 \times 50,580} = 76.2 \text{ lbs./in.}^2$$

2. Under the same assumption as for 1. the average shear below El. 1820 will be

$$S = \frac{407,940,000 + 73,400,000}{144 \times (41099 + 2345 + 2264)} = \frac{481,340,000}{144 \times 45,708} = 73.1 \text{ lbs./in.}^2$$

3. Under the same assumption as for 1. the average shear below El. 1800 will be

$$S = \frac{334,790,000 + 73,150,000}{144 \times (36468 + 2377 + 2254)} = \frac{407,940,000}{144 \times 41,099} = 68.9 \text{ lbs./in.}^2$$

4. Under the same assumption as for 1. the average shear below El. 1780 will be

$$S = \frac{26,3540,000 + 71,250,000}{144 \times (3103 + 31225 + 2140)} = \frac{334,790,000}{144 \times 36,468} = 63.7 \text{ lbs./in.}^2$$

5. Under the same assumption as for 1. the average shear below El. 1760 will be

$$S = \frac{197,840,000 + 65,700,000}{144 \times (3060 + 25745 + 2420)} = \frac{263,540,000}{144 \times 31,225} = 58.6 \text{ lbs./in.}^2$$

6. Under the same assumption as for 1. the average shear below El. 1740 will be

$$S = \frac{138,840,000 + 59,000,000}{144 \times (2508 + 20962 + 2275)} = \frac{197,840,000}{144 \times 25,745} = 53.6 \text{ lbs./in.}^2$$

7. Under the same assumption as for 1. the average shear below El. 1720 will be

$$S = \frac{85,840,000 + 53,000,000}{144 \times (2410 + 16290 + 2262)} = \frac{138,840,000}{144 \times 20,962} = 45.9 \text{ lbs./in.}^2$$

8. Under the same assumption as for 1. the average shear below El. 1700 will be

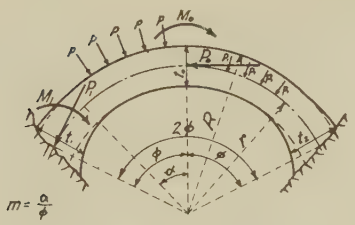
$$S = \frac{38,940,000 + 46,900,000}{144 \times (2488 + 11720 + 2082)} = \frac{85,840,000}{144 \times 16,290} = 36.6 \text{ lbs./in.}^2$$

9. Under the same assumption as for 1. the average shear below El. 1680 will be

$$S = \frac{33,320,000 + 5,620,000}{144 \times (5040 + 3240 + 3440)} = \frac{38,940,000}{144 \times 11,720} = 23.1 \text{ lbs./in.}^2$$

10. Under the same assumptions as for 1. the average shear below El. 1660 will be

$$S = \frac{5,620,000}{144 \times 5,040} = 7.7 \text{ lbs./in.}^2$$



NOMENCLATURE.

The values of C_1, C_2 & C_3 are

$$C_1 = \frac{1.94\phi - 0.47 \sin 2\phi - \sin\phi \cos\alpha}{6\phi + 3\sin 2\phi - \frac{12\sin^2\phi}{\phi}}$$

$$C_2 = \frac{\frac{3\sin^2\phi}{\phi} - \sin\phi\cos\alpha}{6\phi + 3\sin 2\phi - \frac{12\sin^2\phi}{\phi}}$$

$$C_3 = \frac{1.94\phi - 0.47\sin 2\phi}{6\phi + 3\sin 2\phi - \frac{12\sin^2\phi}{\phi}}$$

NOTE—

The calculations are made on the assumption of an arch with fixed ends carrying the total water load with no help from cantilever.

Under the assumptions governing shear calculations we have the approximate increase in shear from the base upwards rather than the exact stress at any single point. In narrow canyons, the halves of the arch partially become horizontal cantilevers through whose vertical sections the load is transmitted by shear to the abutments. This transmission of the stress occurs only with deformation of the cantilever, which deformation is the result of the joint action of arch and horizontal cantilevers.

Reference—"The Circular Arch Under Normal Loads," by Prof. Wm. Cain, Paper No. 1483, Trans. A. S. C. E. 1922.

FIG. 11A

cantilever) without deformation, but the two actions, arch and punching shear, go together and the actual deformation is the result of the joint action. How the load actually divides between the various possible actions towards the bottom is a problem not yet solved by any one, but the principal action presumably is punching shear in connection with some sort of cantilever action both vertical and horizontal. The ultimate crushing strength of the concrete was well above 4000 lb. per sq. in. and the factor of safety should, therefore, be at least five.

The contraction joints, some details of which are shown, were closed somewhat too early. Observation on finished structures has shown that these joints do not open fully for quite a long time after construction is finished. They may show openings on

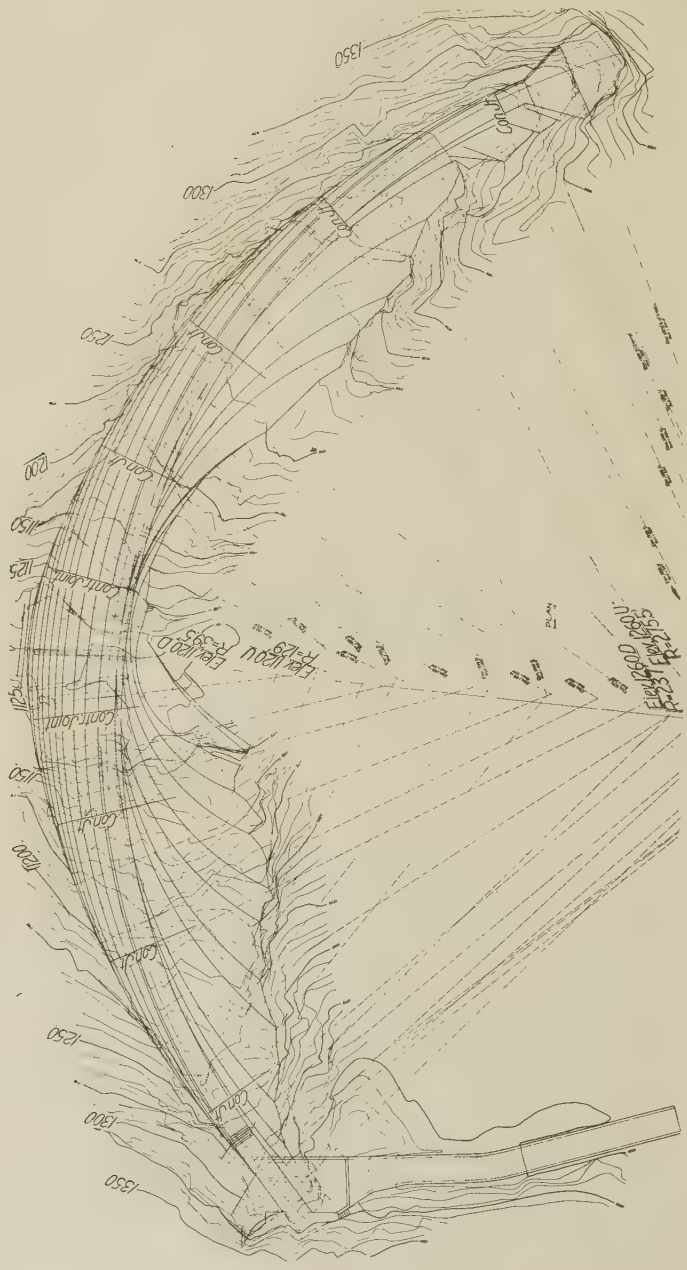


FIG. 12—PLAN—BIG SANTA ANITA DAM

the up and down-stream faces, but farther inside the body where the temperature is highest, due to the presence of the chemical heat and the fact that the structure has not been subject to compressive stresses, the voussoirs still have contact with one another.

If the joints had been grouted during the coldest season after the dam had withstood full water pressure once, the condition of the dam when loaded should correspond to that assumed when making the calculations shown in Fig. 11A. Any increase in temperature will relieve the bending stresses as the arch deflection becomes less with increase in temperature. The condition may actually arise where the expansion of the arch due to increase in temperature is equal to the contraction due to the arch compression in which case there are no bending moments to take care of.

As will be seen from the details, the contraction joints are provided with grout pipes, also with Smith asphaltum waterstops along the upstream face with a copper or galvanized iron stop (grout stop) along the faces. It is between these two stops that the grout is injected. The top of the contraction joint is generally not corked. The grout is injected to just fill the joint to the top. It is not desirable to have the crest of the dam grouted to the extent of exerting a pressure in an upstream direction with a cold dam body inasmuch as the load near the crest is always small compared with the cross section available to support it. At lower elevations there will be grout pressure due to the corresponding static head of the grout.

BIG SANTA ANITA DAM

Figures 12 and 12A show the plan and cross section of the Big Santa Anita dam also built by the Los Angeles County Flood Control District and finished a year ago. Figures 13 and 13A and accompanying table show the calculations. While the tension in the faces is apparently above 100 lb. per sq. in. at elevation 1160, this tension is not paired with a very high compression and this arch slice (and all others below) is located so close to the bottom that it obtains considerable support from punching shear.

Compared with the stresses in the Lake Spaulding Dam, the Santa Anita stresses are not high. Lake Spaulding dam as

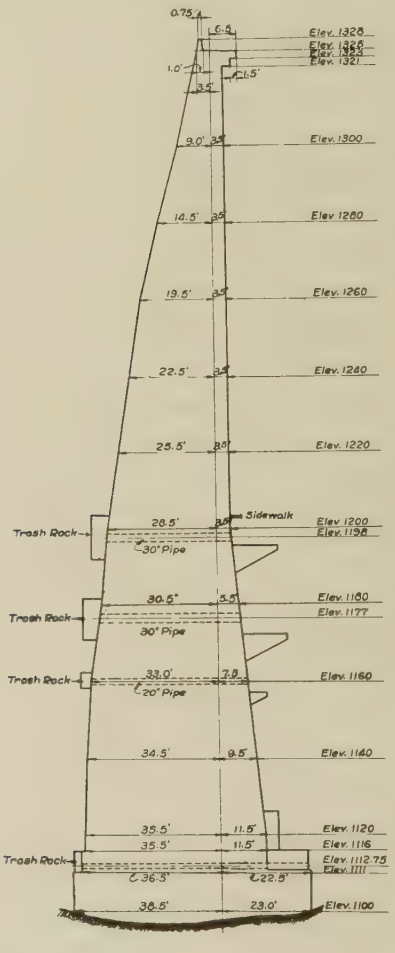


FIG. 12A—MAXIMUM SECTION BIG SANTA ANITA DAM

described in the Transaction of the American Society of Civil Engineers—Volume 78—1915—page 716 was designed at a time, when the simple arch formula was used practically exclusively.

Take for instance—at elevation 4725 the average stress found from the simple arch formula is 308 lb. per sq. in compression. Using the Cain formula, the abutment stresses are found to be

723 lb. compression and 289 lb. tension per sq. in. No particular indication of harm due to this high apparent tension is visible. Very likely the watersoaking effect of the concrete more than compensates for the expansion due to the tension.



FIG. 12B—BIG SANTA ANITA DAM. HEIGHT 225 FT.; SPAN 515 FT.; THICKNESS AT CREST, 7 FT.; AT BOTTOM, 48 FT.

The Big Santa Anita dam is 225 ft. high above stream bed. The length of the upstream radius at the crest is 306 ft. and at the bottom it is 129 ft. long. The span is 515 ft. About 76,000 cu. yd. of concrete went into the dam.

The Pacoima Dam and this dam are thickened gradually from the middle towards the abutments except near the crest. Most

TABLE TO ACCOMPANY FIG. 13

Elevation		1300	1280	1260	1240	1220	1200	1180	1160
R_a	ft.	296.5	287.0	275.5	267.0	256.0	238.0	220.0	190.0
T	ft.	12.5	18.0	24.8	31.6	34.0	37.3	39.8	42.9
P	lbs./ft	1563	2813	4063	5313	6563	7813	9063	10313
$2\,\theta$		117.0°	112.5°	107°	109°	108°	92°	85°	66°
θ		58.5°	56.25°	53.5°	54.5°	54.0°	46.0°	42.5°	33.0°
r	ft	290.5	278.0	256.0	221.0	202.0	168.0	145.0	138.0
$A = \left(\frac{r}{r_1}\right)^2$		540.0976	238.5295	105.5557	48.9114	35.2976	20.2862	13.2730	10.3477
$p \times R_a$	lbs.	463,430	807,331	1,119,357	1,418,571	1,680,128	1,859,494	1,993,860	1,959,470
$p \times R_a \times r$	lbs./ft.	134,626,400	224,438,000	286,555,400	313,385,900	339,385,900	312,395,000	289,109,700	270,406,900
Crown ($m = 0$)	C_1	2.7880	3.0215	3.3459	3.2212	3.2836	4.5475	5.3393	8.9431
	C_2	−0.55269	−0.60194	−0.67039	−0.64406	−0.65722	−0.92476	−1.09025	−1.84756
	C_3	6.1277	6.9463	8.1597	7.6832	7.9146	13.4051	17.5397	42.5473
	$A + C_1$	542.8856	241.5510	109.9016	52.1326	38.5812	24.8337	18.6123	19.2908
	$A + C_3$	546.2253	245.4758	114.7154	56.5946	43.2122	33.6913	30.8127	52.8950
	$K = \frac{A + C_1}{A + C_3}$	0.99388	0.98401	0.95804	0.92116	0.89283	0.73710	0.60405	0.36470
	$P = K \times p \times R_a$	460,594	794,422	1,072,389	1,306,731	1,500,069	1,370,633	1,204,391	714,619
	$M = \frac{C_2}{A + C_1} p \times R_a \times r$	−136,220	−550,352	−1,674,613	−3,567,752	−5,161,764	−8,574,629	−10,229,609	−9,444,995
North Abutment	α_1	55.0°	51.75°	51.5°	46.0°	45.5°	39.5°	30.5°	21.5°
	σ_1	0.94	0.92	0.96	0.845	0.845	0.86	0.715	0.65
	$m = \theta$	4.2171	4.5201	5.1564	4.5890	4.6796	6.5906	7.0302	11.2096
	C_1	0.87642	0.89670	1.14013	0.72377	0.73880	1.11234	0.60065	0.51206
	C_2	544.3147	243 0496	111.7154	53.5004	39.9772	26.8768	20.3032	21 5573
	$A + C_1$	0.99650	0.99012	0.97382	0.94533	0.92514	0.79774	0.65892	0.40755
	$K = \frac{A + C_1}{A + C_3}$	461,808	799,355	1,090,052	1,341,018	1,554,354	1,483,393	1,313,794	798,582
	$P = K \times p \times R_a$	216,008	819,851	2,848,908	4,009,304	5,802,489	10,313,922	5,635,785	2,617,725
	$M = \frac{C_2}{A + C_1} p \times R_a \times r$								

TABLE TO ACCOMPANY FIG. 13 (CONTINUED)

	Elevation	1300	1280	1260	1240	1220	1200	1180	1160
South Abutment	α_2	55.5°	49.25°	45.5°	45.5°	33.0°	32.0°	37.0°	28.0°
	$m = \frac{\alpha_2}{\Theta}$	0.95	0.875	0.85	0.835	0.61	0.695	0.87	0.85
	C_1	4.2448	4.3861	4.7857	4.5589	4.0272	5.8863	7.7983	12.9344
	C_2	0.90414	0.76274	0.76948	0.69373	0.09144	0.42552	1.36871	2.14372
	$A + C_1$	544.3424	242.9156	111.3414	53.4703	39.3248	26.1725	21.0713	23.2821
	$K = \frac{A + C_1}{A + C_3}$	0.99655	0.98957	0.97059	0.94480	0.91004	0.77683	0.68385	0.44016
	$P = K \times p \times R_u$	461,831	798,910	1,086,437	1,340,266	1,528,984	1,444,511	1,363,501	862,480
	$M = \frac{C_2}{A + C_3} p \times R_u \times r$	222,840	697,372	1,922,136	3,842,898	718,164	3,945,538	12,842,346	10,959,007
Crown	$f = \frac{P}{t \times 144} \pm \frac{6 \times M}{t^2 \times 144}$	+292 +220	+377 +235	+455 +191	+568 +126	+615 +103	+646 − 52	+561 − 97	+362Extr. −118Intr.
North Abutment	$f = \frac{P}{t_1 \times 144} \pm \frac{6 \times M}{t_1^2 \times 144}$	+314 +198	+413 +203	+466 +116	+321 +127	+329 +119	+362 + 38	+256 + 88	+147Intr. + 67Extr.
South Abutment	$f = \frac{P}{t_2 \times 144} \pm \frac{6 \times M}{t_2^2 \times 144}$	+315 +195	+398 +218	+326 +160	+295 +129	+284 +246	+313 +143	+529 + 79	+380Intr. −104Extr.

The values of the constants C_1, C_2 and C_3 are as follows:

$$C_1 = \frac{1.94 \times \Theta - 0.47 \sin 2\Theta - \sin\Theta \times \cos\alpha}{6 \times \Theta + 3 \times \sin 2\Theta - 12 \times \frac{\sin^2\Theta}{\Theta}}$$

$$C_2 = \frac{\frac{\sin^2\Theta}{\Theta} - \sin\Theta \times \cos\alpha}{6 \times \Theta + 3 \sin 2\Theta - 12 \times \frac{\sin^2\Theta}{\Theta}}$$

$$C_3 = \frac{1.94\,\Theta - 0.47 \sin 2\Theta}{6\,\Theta + 3 \sin 2\Theta - 12 \times \frac{\sin^2\Theta}{\Theta}}$$

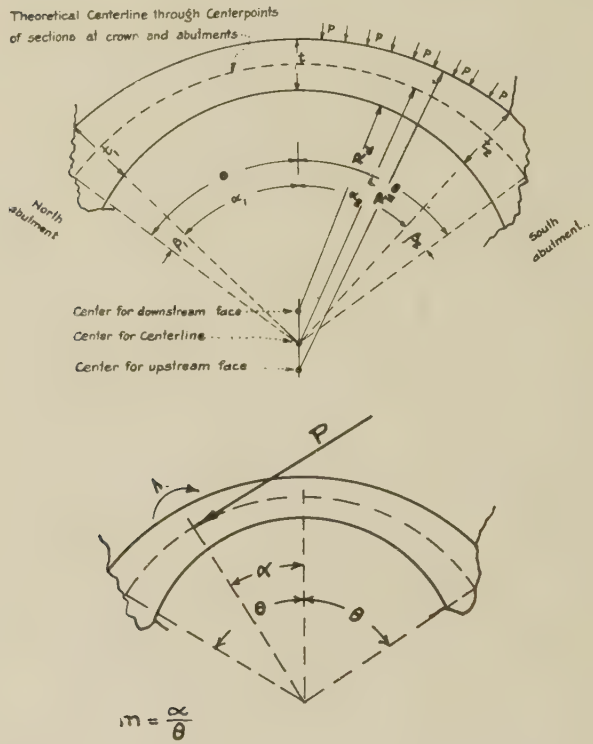


FIG. 13—DIAGRAMS FOR CALCULATIONS FOR BIG SANTA ANITA DAM

TABLE TO ACCOMPANY FIG. 13 (CONTINUED)

	Elev.	1300	1280	1260	1240	1220	1200	1180	1160
Scaled from Drawing	R_u	296.5′	287.0′	275.5′	267′	256′	238′	220′	190′
	R_d	284′	269′	237′	184.5′	160′	120′	107′	86′
	t	12.5′	18′	23′	26′	29′	32′	36′	40.5′
	r	290.5′	278′	256′	221′	202′	168′	145′	138′
	t_1	12.5′	18′	26′	41.5′	48′	51.5′	53′	52′
	t_2	12.5′	18′	31′	44′	40′	44′	42′	43.5′
	$2\,\Theta$	117°	112.5°	107°	109°	108°	92°	85°	66°
	α_1	55°	51.75°	51.5°	46°	45.5°	39.5°	30.5°	21.5°
	β_1	3.5°	4.5°	2.0°	8.5°	8.5°	6.5°	12°	11.5°
	α_2	55.5°	49.25°	45.5°	45.5°	33°	32°	37°	28°
	β_2	3°	7°	8°	9°	21°	14°	5.5°	5°
	T	12.5′	18′	24.8′	31.6′	34.0′	37.3′	39.8′	42.9′
	P	1,563	2,813	4,063	5,313	6,563	7,813	9,063	10,313

Average theoretical thickness $T = \dfrac{4\,t + t_1 + t_2}{6}$

other Constant Angle Arches have been thickened close to the abutments only.

The Ross Construction Co., Los Angeles, were the low bidders and obtained the contract at the following unit prices:

60,000 cu. yd. of excavation—mixed	@	$ 5.50
2,000 cu. yd. of rock excavation (no blasting)	@	8.75
70,000 cu. yd. of plain concrete	@	4.65
300 cu. yd. of reinforced concrete	@	15.00
4,000 lin. ft. placing 2 in. grout and drain pipes and fittings	@	.30
700 lin. ft. placing asphaltum filler with steam pipe	@	.50
100 lin. ft. placing steel discharge pipes	@	1.00
1,000 ft. 3 in. grout holes	@	2.00
28,000 cu. yd. excavation for roadway to damsite	@	1.12

Further details can be obtained from B. F. Jakobsen's paper,* also from articles in Western Construction News, Aug. 25, 1926, by J. W. Reagan and by E. Court Eaton in the Nov. 25, 1927, issue.

B. F. Jakobsen was engineer in charge of the Pacoima Dam and the Big Santa Anita Dam until three years ago. R. B. Metcalf was resident engineer on the Big Santa Anita Dam and Kenneth Harrison was resident engineer on the Pacoima Dam.

DIABLO CANYON DAM

A large constant angle arch dam now nearing completion is shown in Figures 14 and 14A, 14B and 14C. This is the Diablo Canyon dam owned by the City of Seattle, Washington. This represents typical construction of a high arch dam spanning a large gap. Two abutments of gravity section are run out to meet the arch in order to cut down the length of the upstream radius. The arch span proper is 540 ft. and the total gap closed has a maximum width of 950 ft. The maximum height above river bed is 363 ft. and the thickness at the bottom 116.5 ft. Length of upstream radius at the crest is 390 ft. and at the bottom 148.6 ft.

The gravity abutments are built to take care of the overflow during high water in the river and the spillway capacity thus obtained is in excess of 100,000 sec. ft. Normally the spillway bays are closed by means of radial gates. A pressure tunnel connects the reservoir with a powerhouse to be constructed about 2000 ft. below the dam.

*"Stresses in thick Arches of Dams," Trans. of Am. Soc. C. E., 1927, V. 90, p. 583.

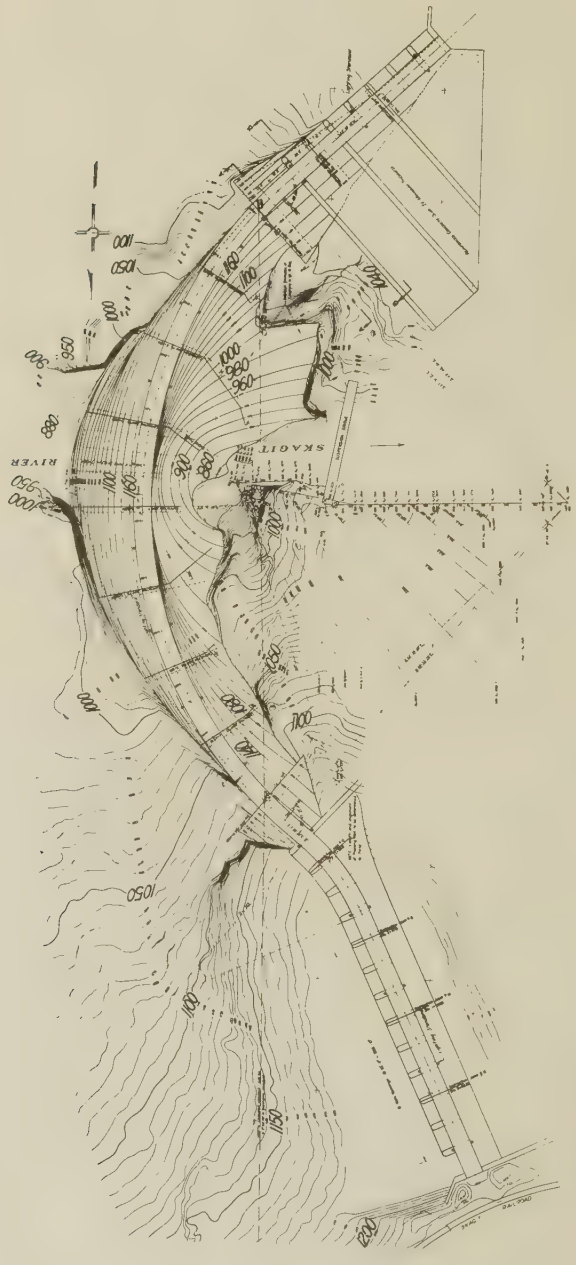


FIG. 14—PLAN OF DIABLO CANYON DAM

Referring to Fig. 14, it will be noted that the gravity abutment has a base thickness of 85 per cent of its height and also that the axis of the abutment is not tangent to the arch crest but inclined from the tangent by an angle of 5 degrees. The abutments are subjected to two sets of external forces: first, the normal water pressure on the upstream face, and second, the resultants of all the arch forces.

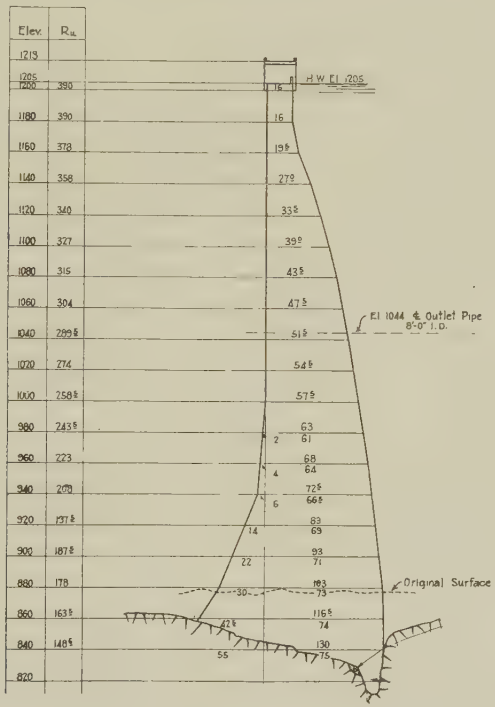


FIG. 14A—MAXIMUM SECTION, DIABLO DAM

The arch resultants may be in turn resolved into their two components, thrust and shear. The arch thrusts are themselves eccentric with respect to the neutral axis of the arch.

In designing the gravity abutments for the Diablo dam it was necessary to find the magnitude and direction of the resultant of all the arch thrusts and radial shears and place the axis of the abutment parallel to this resultant. As a further means of increasing the safety factor of the abutment against overturning,

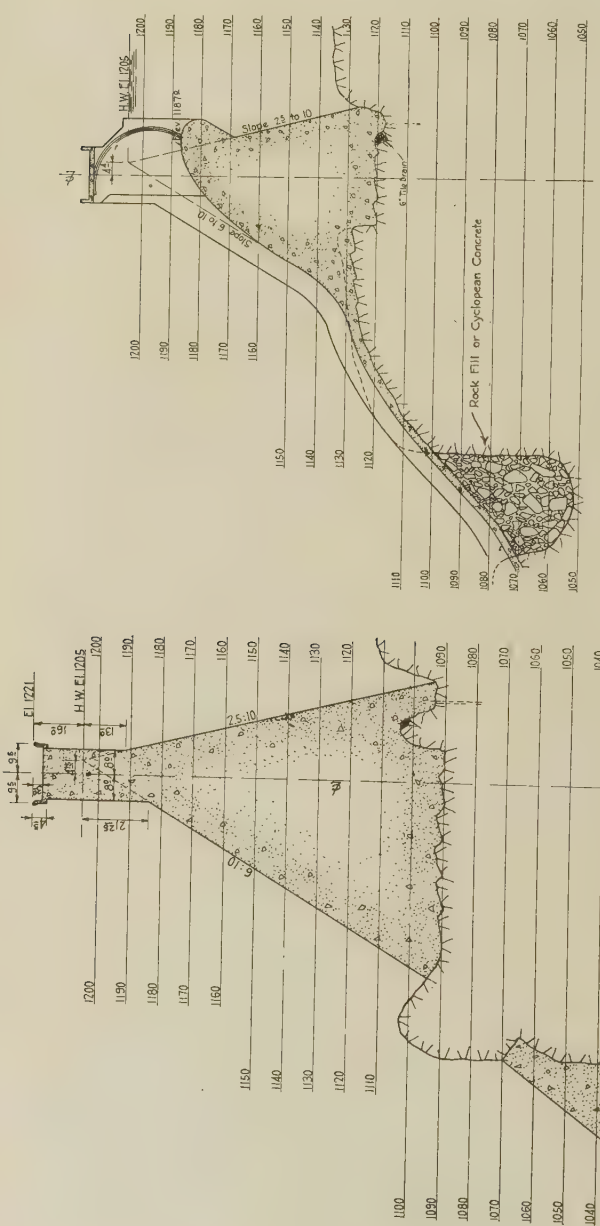


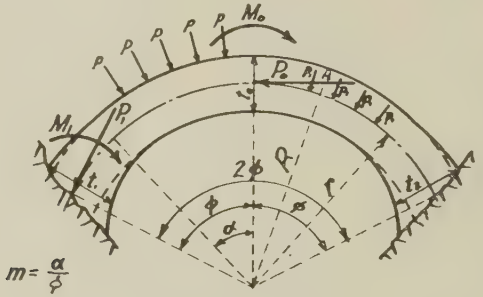
FIG. 14B—SECTIONS, DIABLO DAM



FIG. 14D—DIABLO DAM, 370 FT. HIGH ABOVE FOUNDATION. WIDTH OF GAP CLOSED, 950 FT.; LENGTH OF UPSTREAM RADIUS AT CREST, 390 FT., AT BOTTOM 148.6 FT. 316,000 CU. YDS. OF CONCRETE IN DAM PROPER

in order that it should be comparable to that of the arch itself, the base width of the abutment was made 85 per cent of its height.

As will be observed from Fig. 14, most of the contraction joints are not radial. On account of the fairly large thickness of the arch and the short radius used, this could not be done so well. The grout pipes used in this dam are of an improved type, patented by Arthur Troiel, San Francisco, California, consisting of sheet iron half cylinders and fittings inserted in each face of the contraction joints. When contraction takes place, they offer



NOMENCLATURE.

The values of C_1, C_2 & C_3 are

$$C_1 = \frac{1.94\phi - 0.47\sin 2\phi - \sin\phi\cos\alpha}{6\phi + 3\sin 2\phi - \frac{12\sin^2\phi}{\phi}}$$

$$C_2 = \frac{\frac{\sin^2\phi}{\phi} - \sin\phi\cos\alpha}{6\phi + 3\sin 2\phi - \frac{12\sin^2\phi}{\phi}}$$

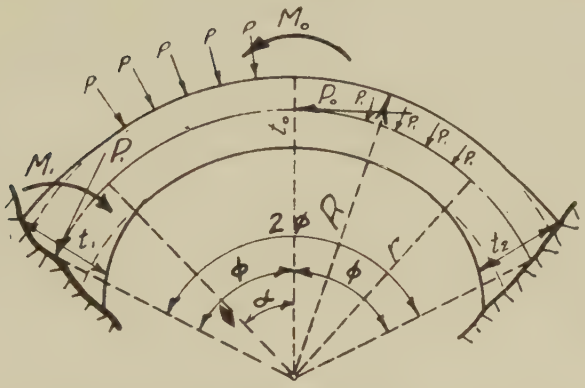
$$C_3 = \frac{1.94\phi - 0.47\sin 2\phi}{6\phi + 3\sin 2\phi - \frac{12\sin^2\phi}{\phi}}$$

NOTE:

The calculations are made on the assumption of an arch with fixed ends carrying the total water load with no help from cantilever.

These calculations are based on actual lines of excavation as of Oct. 15, 1928.

FIG. 15—FINAL DESIGN, DIAGRAM FOR COMPUTATION DIABLO DAM



NOMENCLATURE

Computations of Stresses are made in accordance with Prof. Wm. Cain's Paper No. 1603, Trans. A. S. C. E. 1927, "Stresses in Thick Arches of Dams."

NOTE: These computations are made under the assumption that the arches shall take the *full water pressure* and that arches are fixed at the abutments.

FIG. 15A—FINAL DESIGN. DIAGRAM FOR COMPUTATIONS FOR DIABLO DAM

100 per cent exit opening for the grout as against perhaps 50 per cent in a slotted pipe and they are cheaper to install.

Figures 15 and 15A and accompanying table give the method and result of the stress calculation for the arch down to elevation 920. From and including elevation 960 to the bottom, that is the thick portion of the arch dam, a graphical solution (see also Fig. 15A) for obtaining the arch stresses has been used, as shown on Figures 16, 17, 18, 19 and 20. This method (drawings by G. Goodall) has the advantage that the location of the center line of pressure is visible and if its direction should be unfavorable at the abutments the downstream face of the dam can easily be changed here to improve general conditions at the abutments.

The arch is short and thick below elevation 900 and punching shear stresses are very low even assuming all the load taken up by punching shear. This dam is therefore, exceptionally safe below elevation 900, notwithstanding the fact that apparent arch stresses show up high in the calculations.

Fig. 21, shows the installation of the grout pipes and waterstops on the face of the contraction joint and Figures 22, 22A and 22B give some details of the bridge across the dam. The bridge was designed by Fred Vogt.

TABLE TO ACCOMPANY FIG. 15A

960	940	920	900	880	Elevation
223.0	208.0	197.5	187.5	178.0	R_e (Extrados)
155.0	135.5	114.5	94.5	75.0	R_i (Intrados)
68.0	72.5	83.0	93.0	103.0	$t_o = R_e - R_i$
189.0	171.75	156.0	141.0	125.5	$r = \frac{1}{2}(R_e + R_i)$
186.943	169.167	152.248	135.730	119.172	$r_n = \frac{t_o}{\log_e\left(\frac{R_e}{R_i}\right)}$
34948.807	28614.091	23188.894	18423.990	14202.442	$(r_n)^2$
102°	74°	67°	67°	67°	2ϕ
51°	37°	33.5°	33.5°	33.5°	$\phi°$
.8901183	.6457721	.5846855	.5846855	.5846855	$\phi = .0174533\ \phi°$
.9781575	.9612615	.9205053	.9205053	.9205053	Sin 2ϕ
.2079117	.2756374	.3907311	.3907311	.3907311	Cos 2ϕ
.7771459	.6018151	.5519370	.5519370	.5519370	Sin ϕ
.6293204	.7986354	.8338858	.8338858	.8338858	Cos ϕ
.1269185	.0680690	.0560105	.0560105	.0560105	$1 - \frac{\sin\phi}{\phi}$
1.3791170	1.1264028	1.0449381	1.0449381	1.0449381	$\phi + \frac{1}{2}\sin 2\phi$
.4010395	.1651413	.1244328	.1244328	.1244328	$\phi - \frac{1}{2}\sin 2\phi$
1.3570238	1.1217000	1.0420455	1.0420455	1.0420455	$1 - \frac{\cos 2\phi}{\phi}$
1.0110033	1.0152689	1.0246440	1.0388271	1.0614909	$\frac{r}{r_n}$
.0110260	.0153060	.0247669	.0391231	.0822487	$\left(\frac{K}{r_n}\right)^2 = \frac{1}{12}\left(\frac{t_o}{r_n}\right)^2$
.0502553	.0293343	.0378668	.0583386	.0916182	$D_n - (\phi + \frac{1}{2}\sin 2\phi)\left(1 + \frac{K^2}{r_n{}^2}\right) - \frac{1 - \cos 2\phi}{\phi} + 2.88\ \frac{r}{r_n}\left(\frac{K^2}{r_n}\right)(\phi - \frac{1}{2}\sin 2\phi)$
30.927918	41.03149	29.15149	18.92184	12.04863	$\frac{2\sin\phi}{D_n}$
15,313	16,563	17,813	19,063	20,313	p
3,414,799	3,445,104	3,518,068	3,574,312	3,615,714	$\mu \times R_i$

TABLE TO ACCOMPANY FIG. 15A (CONTINUED)

1,164,485	2,163,622	2,540,017	2,645,995	2,711,827		$p\,R_e - P_o = p\,R_e \frac{2\sin\phi}{D_e}\left(\frac{K}{r_n}\right)^2$
2,250,314	1,281,482	978,051	928,317	903,887		$P_o = p\,R_e - p\,R_e\left[\frac{2\sin\phi}{D_e}\left(\frac{K}{r_n}\right)^2\right]$
−27,629,182	−24,914,170	−21,659,961	−20,115,662	−18,101,128		$M_o = -(p\,R_e - P_o)\,r_n\left(1 - \frac{\sin\phi}{\phi}\right)$
2,681,965	1,717,159	1,399,984	1,367,854	1,354,360		$P_1 = p \times R_e - (p\,R_e - P_o)\cos\phi$
53,064,919	48,787,980	42,578,478	39,542,741	35,582,638		$M_1 = M_o + (p\,R_e - P_o)\,r_n\,(1 - \cos\phi)$
2.057	2.583	3.752	5.270	7.328	Crown	$C = r - r_n$
26,203	31,756	47,649	67,030	91,060		$I = {}^1/_{12}\ t_o^3$
+414	+272	+173	+128	+95		Extrados: $S = \frac{P_o}{t_o \times 144} - \frac{M_o + P_o\,C}{I \times 144} \times \frac{r_n}{R_e}\left(\frac{t_o}{2} + C\right)$
−5	−76	−50	−24	−0.5		Intrados: $S = \frac{P_o}{t_o \times 144} + \frac{M_o + P_o\,C}{I \times 144} \times \frac{r_n}{R_i}\left(\frac{t_o}{2} - C\right)$
10	2.5	2	0	0	North Abutment	Upstream Add = a =
24	12	4	3	1		Downstream add = b =
102	87	89	96	104		t_1 =
88,434	54,875	58,747	73,728	93,739		$I_1 = {}^1/_{12} \times t_1^3$
+393	+462	+408	+366	+329		Intrados: $S_i = \frac{P_1}{t_1 \times 144} + \frac{M_1 + P_1\left(C + \frac{a-b}{2}\right)}{I_1 \times 144} \frac{r_n}{R_i}\left(\frac{t_o}{2} - C + b\right)$
−18	−55	−91	−59	−40		Extrados: $S_e = \frac{P_1}{t_1 \times 144} - \frac{M_1 + P_1\left(C + \frac{a-b}{2}\right)}{I_1 \times 144} \frac{r_n}{R_e}\left(\frac{t_o}{2} + C + a\right)$
13	5	0	0	2	South Abutment	Upstream Add = a =
25	28.5	29	24	18		Downstream Add = b =
106	106	112	117	123		t_2 =
99,251	99,251	117,077	133,468	155,072		$I_2 = {}^1/_{12} \times t_2^3$
+380	+292	+231	+229	+ 230		Intrados: S_i
−14	+30	+30	+22	+13		Extrados: S_e

The reservoir will store 90,000 acre ft. of water and will regulate the Skagit River to a certain extent for the benefit of an existing power-plant located approximately eight miles below.

In the near future a large power plant will be finished below the Diablo Dam utilizing about 335 ft. head.

This dam is being constructed by Winston Bros., Minneapolis, who were low bidders at the following unit prices. Cement furnished by the City of Seattle at $2.50 per bbl. is included in the bid:

River diversion and control—lump sum	@	$190,000.00
63,000 cu. yd. solid rock excavation above elv. 900	@	3.50
3,000 cu. yd. solid rock excavation below elv. 900	@	6.75
7,000 cu. yd. solid rock excavation in tunnel and shaft	@	6.75
12,000 cu. yd. of earth and loose rock excavation	@	3.50
210,000 cu. yd. of class "A" concrete	@	6.97
20,000 cu. yd. of class "B" concrete	@	7.40
12,000 cu. yd. of class "C" concrete	@	9.25
20,000 lb. of copper waterstops	@	.53
1,600 ft. asphaltum in contraction joints	@	1.73
2,000 barrels of cement for grout	@	7.20
Total cost of dam—approximately		$3,000,000.00

There is a bonus and penalty clause in connection with this estimate. It is desired to have the dam constructed to elevation 1050 as soon as possible in order to make some storage available for the lower power plant. The contractor receives a bonus of $1,000.00 per day for each day he completes the dam to this elevation sooner than 366 days from the day he was told to go ahead. He pays a penalty of $1,000.00 a day for each day he runs over this period.

It is desired to have the dam completed in 731 days and there is a clause in the contract providing for a bonus or penalty of $200.00 per day in this connection.

Gravel of a good grade is available within about 1000 ft. of the dam. At this place is also the terminus of a standard gauge railway built by the City of Seattle for hauling cement and other material to the dam site.

The major portion of the concrete in the dam contains 5 sacks cement per cubic yard but the upstream face of the dam to a depth of 4 ft. contains 6 sacks per cu. yd. The City of Seattle maintains a laboratory adjoining the construction camp and conducts sieve analyses of all aggregates. The exact proportion of the mix is determined as a result of these tests and careful

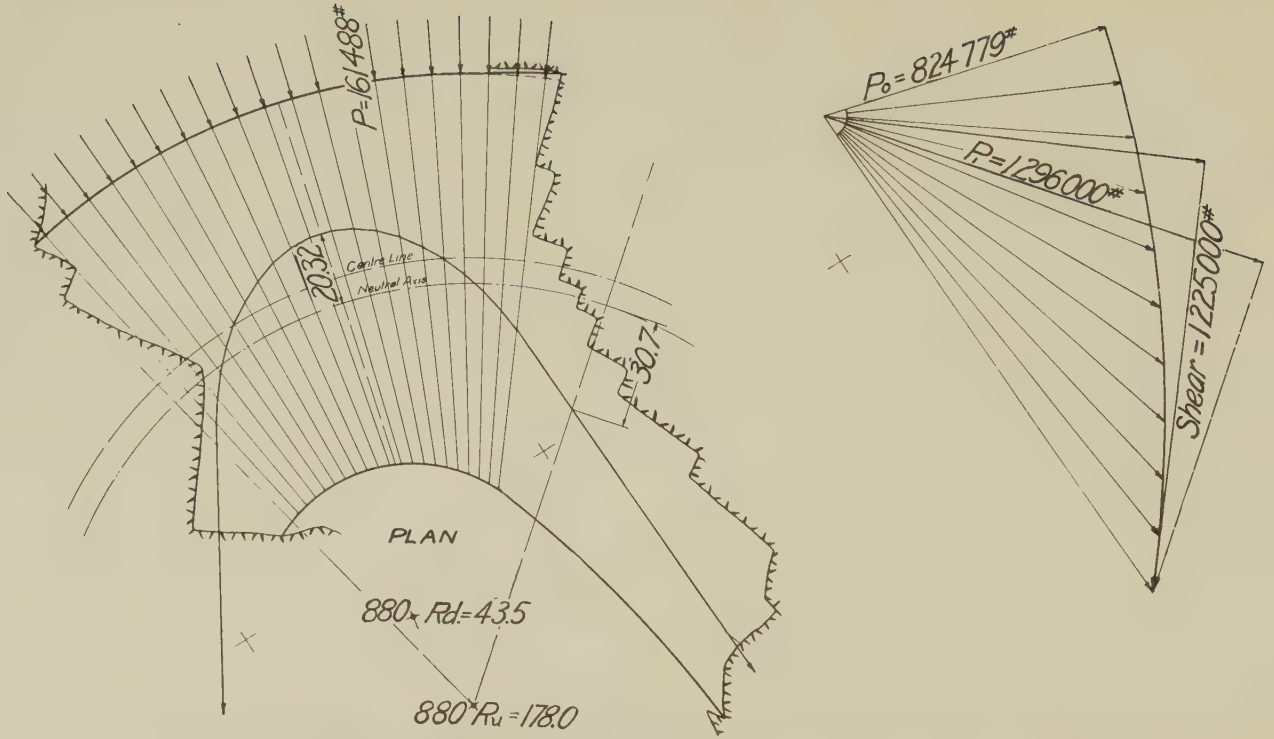


FIG. 16—DIABLO CANYON DAM. PRESSURE DIAGRAM FOR ELEVATION 880

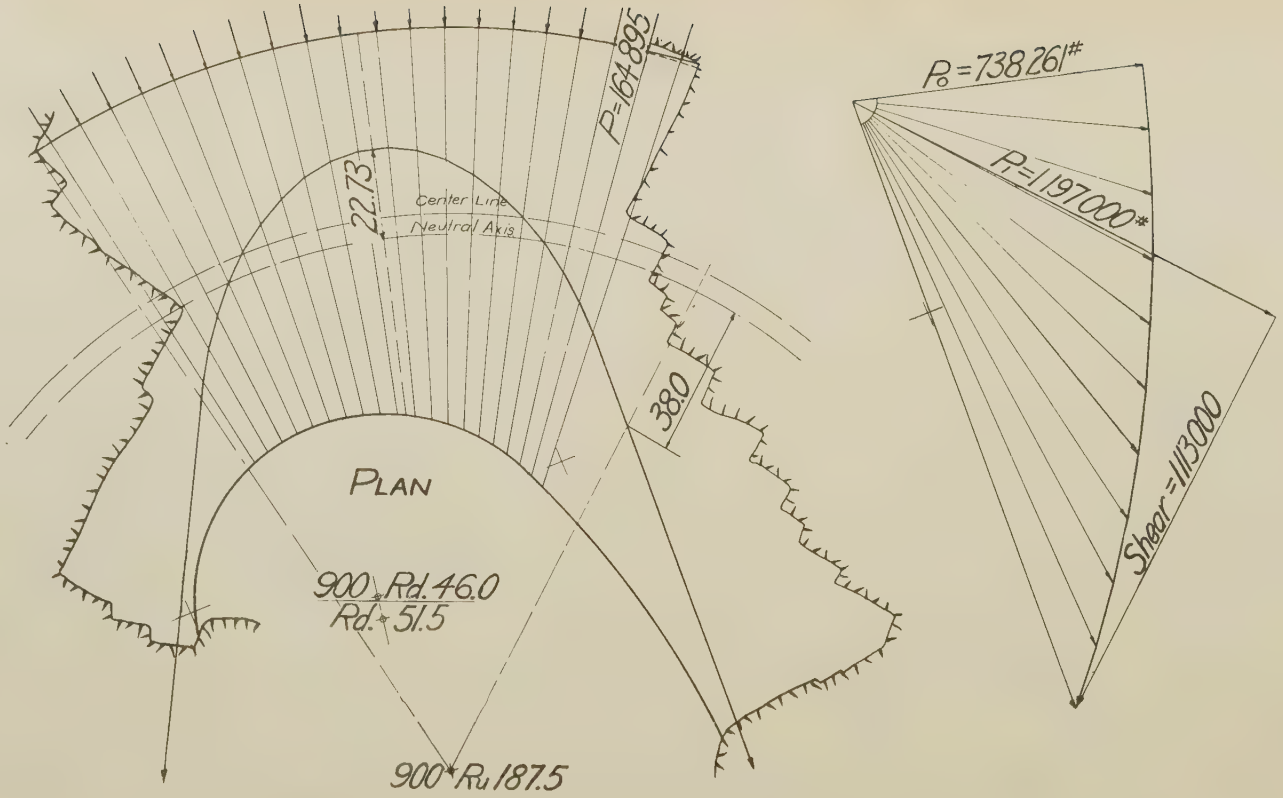


FIG. 17- DIABLO CANYON DAM. PRESSURE DIAGRAM FOR ELEVATION 900

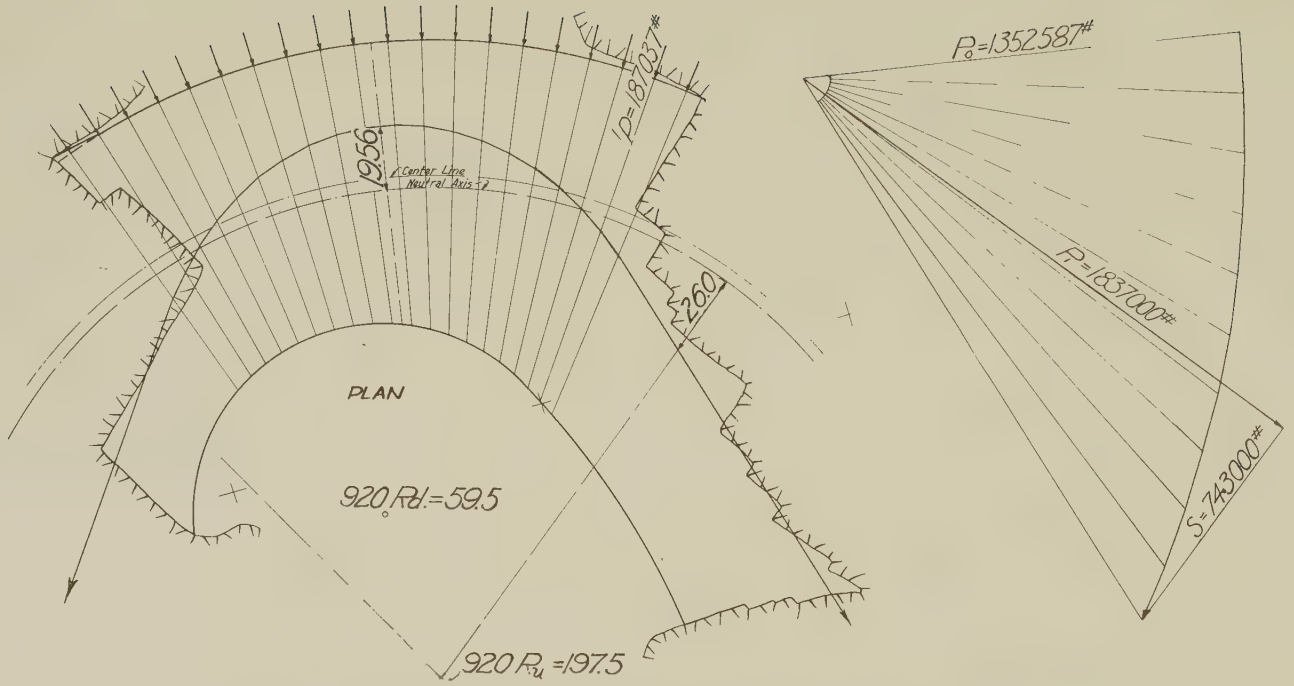


FIG. 18—DIABLO CANYON DAM. PRESSURE DIAGRAM FOR ELEVATION 920

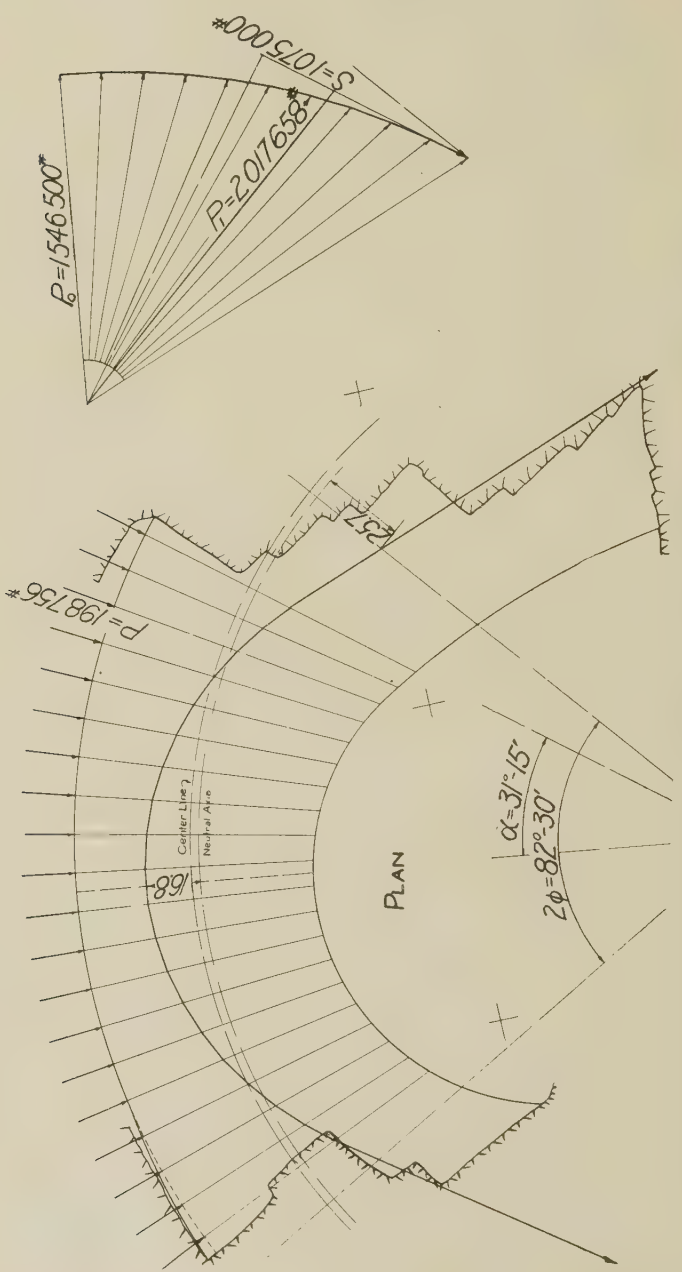


FIG. 19—DIABLO CANYON DAM. PRESSURE DIAGRAM FOR ELEVATION 940

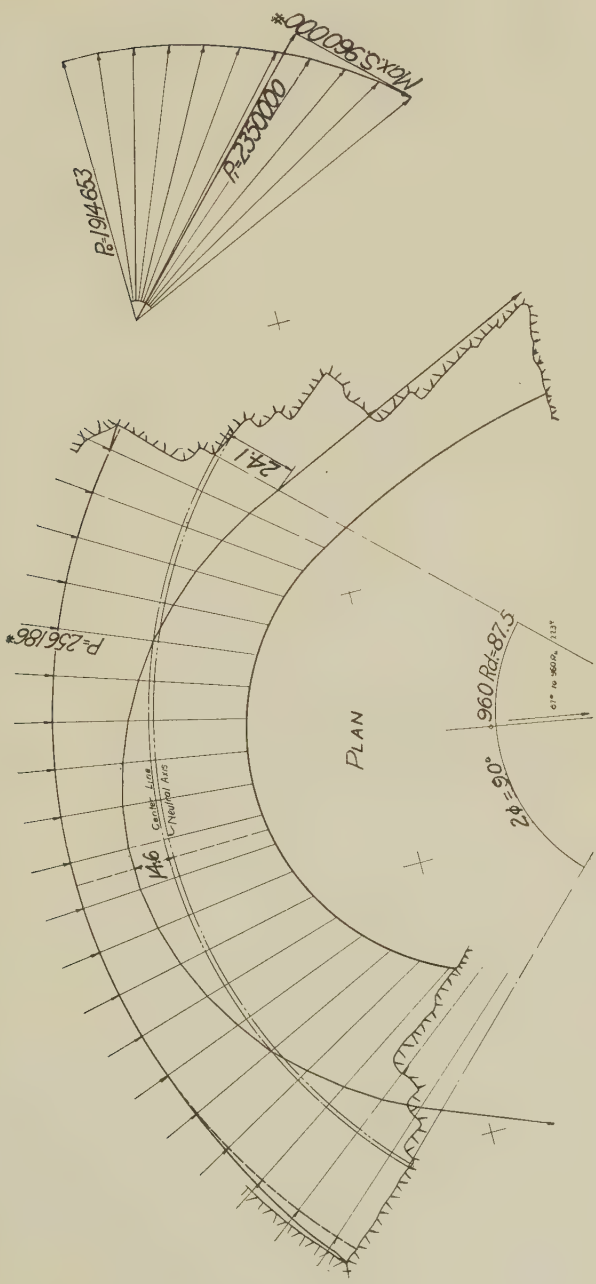


FIG. 20—DIABLO CANYON DAM. PRESSURE DIAGRAM FOR ELEVATION 960

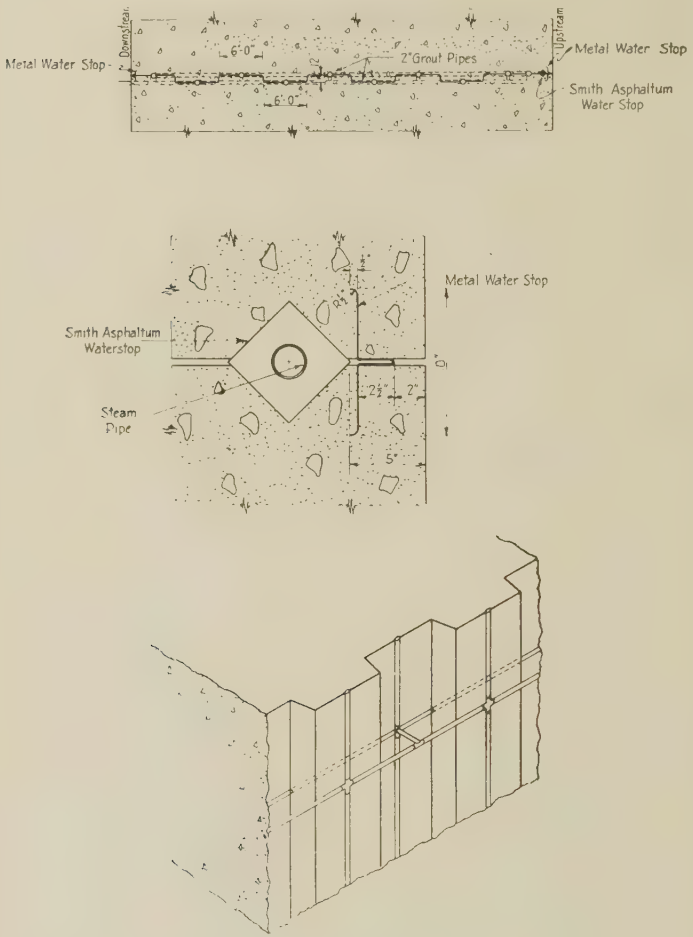


FIG. 21—DETAILS OF WATER STOP, DIABLO CANYON DAM, AND SKETCH SHOWING ARRANGEMENT OF PIPES

control is maintained over the water-cement ratio. As a result of the care in proportioning the mix and the exceptionally fine quality of the aggregates, the concrete develops high crushing strength in 28 days. The general average compressive strength in 28 days is about 4200 lb. per sq. in. The maximum compressive stress in any portion of the dam as computed by Cain's analysis is 599 lb. per sq. in.

The effect of swelling due to water soaking was investigated in connection with the design of this dam. The calculation of these stresses was made by use of the formulas derived by B. F. Jakobsen in his paper on "Stresses in Thick Arches of Dams" and published in Vol. 90 Trans. Am. Soc. C. E.

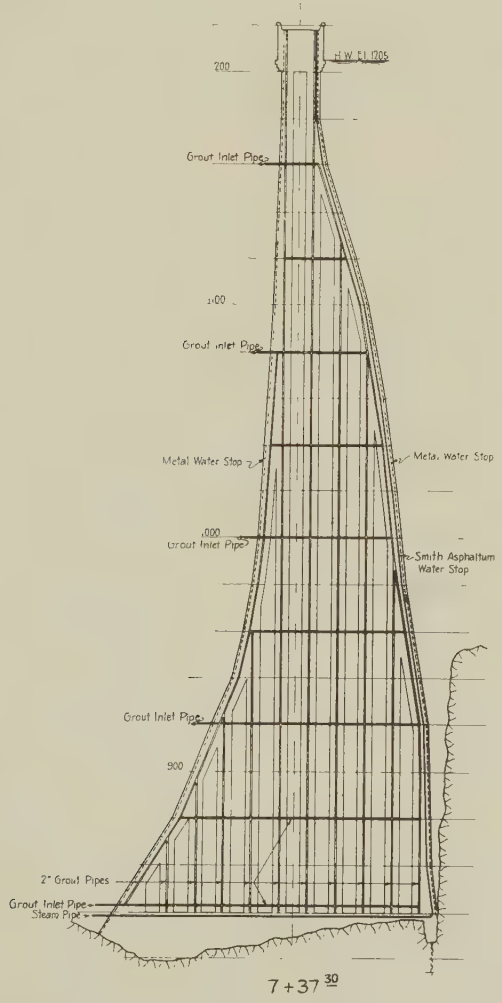


FIG. 21A—INSTALLATION OF GROUT PIPES AND WATER STOPS, DIABLO CANYON DAM

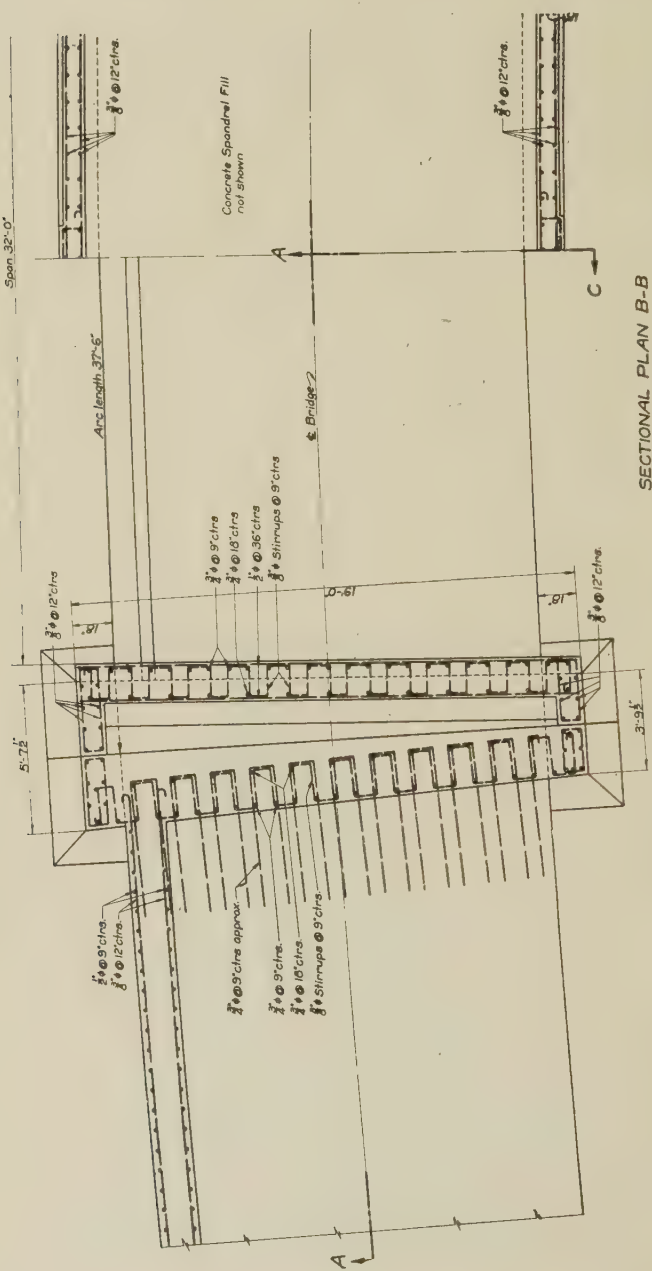


FIG. 22—DIABLO CANYON DAM. SECTIONAL PLAN OF ARCH BRIDGE

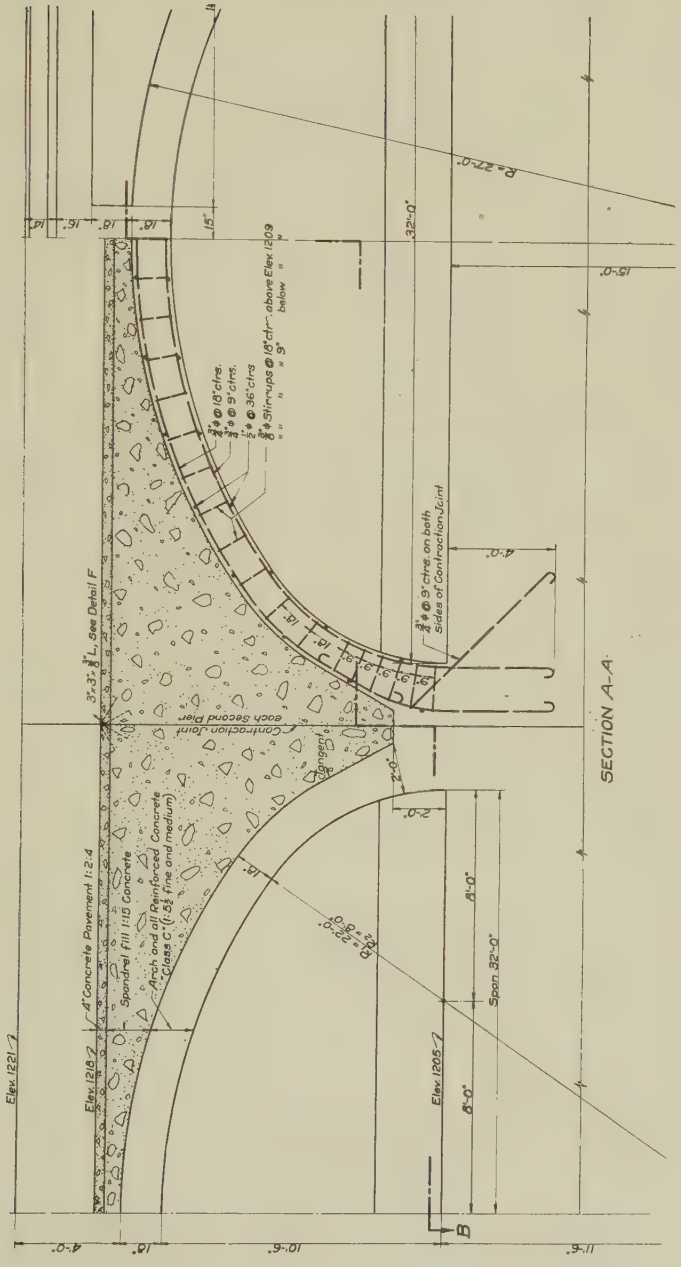


FIG. 22A—DIABLO CANYON DAM. DETAILS OF ARCH BRIDGE

For the Diablo Dam, the calculation of stresses due to swelling showed small increases in the compressive stresses at the crown section values of thickness divided by mean radius less than 0.25 and for ratios greater than 0.25, greater increase of compressive stresses at the extrados accompanied by a lowering of the compressive stress at the intrados, or if Cain's analysis indicated tension at the intrados, then an increase in the tension.

At the abutments the effect of swelling was much greater, the bending moments being large and of opposite sign to the moments due to water load on the arch. The general effect was to cause a very material decrease in compression at the intrados and an increase in compression at the extrados so that if Cain's analysis indicated tension at the extrados, the stresses due to swelling being of opposite sign, the resulting stress became compression.

For numerical values of all stresses in Diablo Dam and a detailed description of the dam and appurtenances refer to "Diablo Dam" by G. E. Goodall in Western Construction News, May 25, 1929.

The Constant Angle Arch Dam Co. furnished the design and acts as consulting engineers. City Engineer W. C. Morse, had charge of the development at the start. Later, W. D. Barkhuff was appointed city engineer and took over Mr. Morse's duties. T. H. Cover was his original assistant in charge of the work.

It is expected to finish this dam in the fall of 1930.

When looking back over the various structures described in the foregoing, it will be noticed that no regularity about the spacing of the contraction joints exists.

The writer has not yet found any spacing which he thinks is the best in general.

To be sure that no vertical cracks would appear between the artificial contraction joints, it would be necessary to have a joint every 40 ft. (or perhaps 50 ft. would still be safe). Since each joint costs a comparatively large amount of money the desire for increasing this distance is apparent and it will be noted that the spacing of the contraction joints on the dams illustrated in the foregoing is nearer twice 40 ft. than 40 ft.

On a "round" slender arch dam this spacing will generally still be safe, without any other cracks developing later, but on some of the heavier arches some chance of a vertical crack appearing

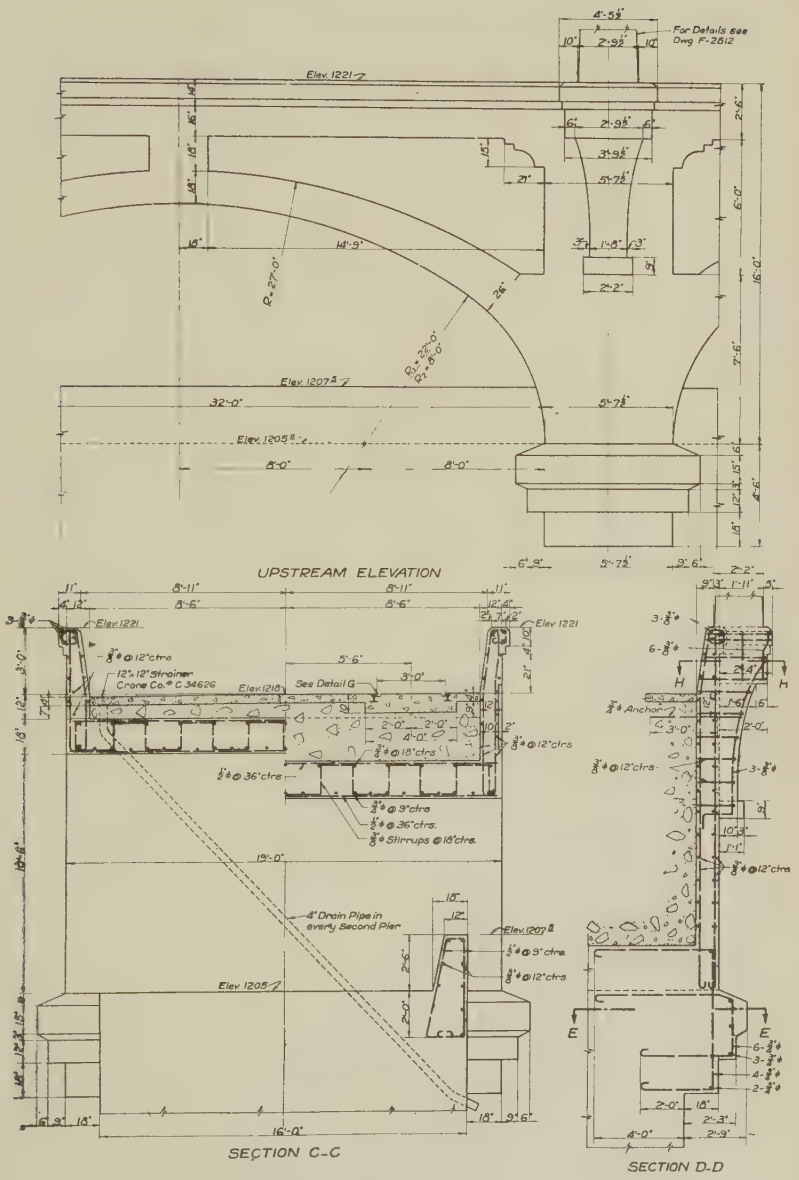


FIG. 22B—DIABLO CANYON DAM. DETAILS OF ARCH BRIDGE

between two contraction joints has been taken in order to lessen the joint cost. It is felt that such a crack will in each instance close water tight when the regular contraction joints are grouted.

On the Lake Spaulding dam previously mentioned such vertical cracks did appear very definitely, but the writer recalls but few other instances of pronounced cracks between artificial joints spaced over 50 ft. apart.

When a reservoir is filled for the first time, the dam or the foundation or mostly both will be found to leak some. Some of these leaks will close up by themselves but most of them will need grouting. It is not difficult to make a concrete dam and especially an arch dam commercially water tight by grouting. Theoretical water tightness can not be obtained, at least the writer has not observed this condition as yet, but it is important to carry on the grouting work to a considerable extent, when a dam is first put into use. The tighter the concrete dam the longer it can be expected to exist. Some owners or their engineers are rather reluctant in carrying out this process of water tightening after the structure is finished. They will go and look at some other dam, which leaks worse than theirs and then be satisfied with their own in a state not close enough to perfection.

It will perhaps be noted that temperature stresses have not been included in the calculations of the dams described. More importance is attached to the grouting of the contraction joints at a time when the dam body has its minimum temperature or at least as near the minimum as practicable. After this is done there can be no additional temperature stresses with water in the reservoir. With rising temperature the stresses will be less than calculated since the expansion due to the higher temperature will compensate in a measure for the rib-shortening whereby the bending stresses become smaller or may be entirely eliminated. If a reservoir is being filled gradually during the spring and early summer, it is not uncommon to observe the dam (arch dam) deflecting only about one third of what it would have deflected had the temperature remained constant and a minimum during the time of filling.

Readers are referred to the JOURNAL *for January, 1931, for discussion which may develop. Such discussion should reach the Secretary by December 1, 1930.*

Construction Specifications For Concrete Work on the Small Job

Proposed American Concrete Institute Specification No. 506

ARTHUR R. LORD,* AUTHOR-CHAIRMAN
Committee 506—Concrete Specifications for the Small Job.†

This proposed specification is unusual in conception and treatment. It may be read straight through on right-hand pages only, but is supplemented by "Specification Notes" appearing on left-hand pages and keyed to the specification proper by note numbers.—Editor

TO THE USER

This specification is intended for the use of Architects and Engineers, to be incorporated as a whole in the specifications for particular structures by reference in the proper place in the typewritten pages, and to become a part of the contract in the usual manner. It is intended for use on the *small job* on which full time inspection is not maintained, and it is based on the assumption that such work will be awarded by the Architect or Engineer *to honest contractors only* who can be depended upon to do high grade work without close inspection. This specification attempts to explain to the less experienced contractor, by means of numerous notes, why the work must be done in the specified manner. It assumes that poor concrete work on small jobs is due to lack of technical knowledge rather than to dishonesty and that the contractor, in preparing an estimate under such a specification as this is, will understand what is required of him to do good concrete work and will submit his bid on the basis

*President, Lord and Holinger, Inc., civil and architectural engineers, Chicago.
†Members: Theodore Crane, P. J. Freeman, S. C. Hollister, W. F. Way.

Specification Notes. *(See Specification Text Opposite)*

GENERAL NOTE ON CONCRETE MAKING

The major factor in determining the degree to which any concrete will develop strength, watertightness, and many other desirable properties, is the *ratio of the water to the cement* in the concrete as finally deposited in the forms. This basic relation was first reported by Duff A. Abrams in 1915, and has been exhaustively verified by many other experimenters. It has been shown to apply to concretes having strengths ranging all the way from 200 lb. per sq. in. to over 50,000 lb. per sq. in. Specification of concrete quality in terms of water-cement ratio (generally expressed as number of *gallons of water per sack of cement*) is now almost universal on larger work and is the only safe and intelligent basis of specification for small jobs also.

The time honored specification by loose volumes, e. g. one part cement to two parts sand to four parts gravel (or stone), is definitely obsolete; 1:2:4 concrete may develop a strength of 5000 lb. per sq. in. when used with a low water-cement ratio such as is employed in tamped paving bases, or it may develop less than 10 per cent of that strength when used with excess mixing water. The use of a *water-cement ratio* type of specification releases the contractor from fixed arbitrary proportions of sand and gravel (or stone). Under this type of specification the proportions of fine and coarse aggregate may be varied at will by the Contractor, during the time used to determine upon the mix to be used on any job, so as to produce the maximum economy obtainable without sacrificing the necessary ease in placing the concrete. This best proportion, as thus determined from trial batches (see Article 10), will vary widely with different fine and coarse aggregates and is rarely found to be in the old ratio of two to four.

A "philosophy" of concrete making—and what is more important, a practical comprehensive discussion of the entire subject—may be found in a small book by Franklin R. McMillan, "Basic Principles of Concrete making."*

Concrete is a mass of inert aggregates held together by hardened paste of portland cement and water. The properties of the paste—strength, water-

*McGraw Hill Book Co., $2.00. The following extracts from an abstract of this book (*News Letter*, JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Dec. 1929) indicate basic considerations developed fully in the book.

(*Notes Continued Page 68*)

of following these instructions, even though not forced by supervision to do so. While this is an unusual specification, as regards these explanatory and educative notes, it is obvious that the only assurance of quality on a job subject only to casual and occasional inspection lies in the character of the contractor himself. When the Architect or Engineer has discovered his honest contractor, it is believed that this type of specification will materially help in securing fine concrete work. If the contractor is dishonest, no form of specification will prevent poor work in the absence of effective and continual supervision.

The following wording is suggested to be used in the Architect's and Engineer's typewritten specification for individual buildings:

Specification for Concrete Work
Name and Location of Building
Name and Address of Owner
Name and Address of Architect or Engineer

All plain and reinforced concrete work on this structure shall be done in strict accordance with Specification No. 506 of the American Concrete Institute, which is hereby made a part of these specifications and which will be made a part of the contract for this work. All the provisions of Specifications No. 506 shall be in full force and effect, except as expressly modified or supplemented in the following special instructions for this structure.

(Here incorporate any clauses required or suggested by unusual conditions surrounding the work to be done.)

Wherever the words "Architect or Engineer" are used the same shall be construed to mean the Architect Engineer (strike out one) only.

This specification has been submitted to the following critic members of Committee 506, who have greatly assisted the author by their thorough cooperation and experienced criticism:

Theodore Crane, Professor of Architectural Engineering, Yale University, New Haven, Conn.

P. J. Freeman, Dept. of Public Works, Allegheny Co., Pittsburgh, Penna.

S. C. Hollister, Consulting Engineer, Swarthmore, Penna.

W. F. Way, Henry and McFee Contracting Co., Contractors and Engineers, Seattle, Washington

(*Specification Notes—Continued from Page 66*)

tightness, durability, etc.—determine the properties of the concrete. The quality of paste depends upon characteristics of cement; upon water-cement ratio; and upon time, favorable temperature and continued presence of water to complete hydration of the cement.

A plastic, homogeneous mixture is vital to good concrete making, meaning a mixture in which the constituent materials are evenly distributed in all parts of the mass as finally placed and hardened and one which, when freshly mixed, flows sluggishly but without segregation of water or fine materials from the coarse.

Plasticity depends upon relative quantities of paste and aggregate, upon plasticity of the paste itself, upon grading of aggregates and upon shape and surface characteristics of aggregate particles. The amount of paste must be sufficient completely to fill the voids in the aggregates and to float the aggregate particles. The paste must not be so diluted with water as to be unable to hold the particles in a cohesive mass nor so stiff as to make whole mass difficult to handle and place. Grading of aggregates determines quantity of paste required to float particles and also resistance to ready motion or flow of the aggregate itself. Shape of aggregate particles likewise affects the resistance to flow of the aggregate and of concrete made from it. Surface characteristics affect readiness with which paste surrounds and firmness with which it holds the particles and amount of paste required for plastic concrete.

A portion only of the water used in mixing concrete combines chemically with the cement—if only enough water were used to hydrate cement, concrete would be entirely unplaceable. Remainder of water—commonly the greater part of water used—makes it possible to handle the concrete but is also responsible for most of later difficulties. The greater the amount of uncombined water the less the water-tightness of the concrete. Additional curing causes more water to combine with cement.

Very great increases in strength are secured by continued curing. Curing is a major factor in concrete strength and its importance is not sufficiently realized on job. Age, with continued curing, shows consistent gain in strength following a W/C ratio law at all ages. The superiority of concrete with low W/C ratio over that with high W/C ratio increases with age.

A thorough study of this book will assist a Contractor to produce better concrete at no increase in cost.

Contractors who, for business reasons, deal entirely with one material dealer and who can obtain the same cement and aggregates on much or all of their concrete work, will do well to have experiments made to determine the water-cement-ratio-strength law for that particular combination of materials. For it must be recognized that the materials used affect the strength of concrete. In the absence of such advance tests the approximate strength of *well-cured plastic homogeneous concrete*, made from

(*Notes Continued Page 70*)

1. General conditions of the contract

This Contractor shall acquaint himself fully with the General Conditions of the Contract for this work. He shall be bound by all requirements of these general conditions insofar as they are applicable to his part of the work in the opinion of the Architect or Engineer. These General Conditions shall be made a part of the contract for the concrete work covered by this specification.

The Contractor shall read this Specification with the greatest care, taking into consideration all specification notes appearing herein, and shall carry on the work in accordance with the letter and spirit of this specification at all times without regard to the presence or absence of the inspector.

2. Scope of the concrete work

This Contractor shall provide all necessary labor and material and install complete, ready for convenient use, all portions of the structure specifically mentioned in these specifications and/or shown on the architectural or engineering plans or details of the structure, and also all usual necessary and essential portions of a structure of this type, that would commonly be made of plain or reinforced concrete, whether or not the same are definitely shown on the plans or listed in the specifications. In general the work of this Contractor shall include: concrete foundations; concrete floors on the ground, including fine grading and filling therefor; concrete retaining walls at building lines and area ways; concrete columns, roofs and floors; concrete walls; concrete or terrazzo floor or stair finish; any cutting, shoring, sheeting, hand excavation, backfilling or underpinning necessary to install concrete work properly; concrete sidewalks, paving and parking strips; installation of masonry ties or other inserts; concrete steps and stairways; concrete penthouses or tank supports; formwork for all concrete work requiring same; and so forth. This contractor shall also furnish and place all necessary reinforcing material, properly bent and marked, together with complete accessories to securely hold all reinforcement in its designed position during construction.

(*Specification Notes—Continued from Page 68*)

ordinary materials, may be taken as stated in article 10. Certain brands of cement develop strength much more rapidly than others, (*see Note 3*) and concrete made of them will have higher early strength. Crushed stone has been found to give higher strengths than gravel, with the same water-cement ratio. This apparent gain is almost exactly neutralized in the average case, however, by the fact that more paste is required to make crushed stone concrete workable than is required for gravel concrete. In much the same way finer aggregate seems to give higher strengths than coarser aggregate at the same water-cement ratio, but here again more paste is required to make the concrete thoroughly workable as the aggregate becomes finer.

For practical purposes the water-cement-ratio-strength law is limited, in its application to actual construction work,

(1) to plastic (readily placeable, not over-dry nor over-wet) mixes (see abstract reprinted above)

(2) to favorable conditions on the job that provide ample moisture in the concrete for the continued hydration of the cement (*see Note 11*)

(3) to favorable temperatures on the job. The rate of gain in strength of concrete is very much less at low temperatures than at high temperatures, becoming almost nil at temperatures below 40° F. (*see Note 19*).

Each subsequent note applies to that portion of the specification after which the same reference number appears.

Note 1

It is rarely possible to use sand from the excavation, on account of the high probability that dirt will become mixed with or may be already contained in it, in injurious quantities. This is a very frequent source of trouble, and aggregates from the site must not be used, except with the Architect's or Engineer's permission and only after exceptionally thorough tests.

Stone that deteriorates on exposure to the weather, or from pits or quarries where there is evidence of such deterioration, is unsafe to use.

Note 2

For sand for mortar topping (floors, sidewalks, etc.) not over 10 per cent shall pass the No. 100 sieve.

3. Conditions at site of work

This Contractor shall visit the site of the work and make himself thoroughly familiar with all conditions affecting the conduct of his work, the location and arrangement of his plant, the storage of materials, and so forth. He shall make any necessary observations as to soil conditions, ground water level, sewers or public utilities, traffic requirements and so forth. No additional allowance of time or money shall be asked or granted because of this Contractor's lack of knowledge of conditions prevailing at the site at the time of estimating, or which may be readily foreseen to exist at the time that the work is undertaken.

4. Concrete materials

Portland cement shall be used for all concrete work. This cement shall be certified by the dealer to comply with all requirements of the standard specifications of the American Society for Testing Materials. (Serial designation: C9-26, or latest standard of this Society.)

Aggregate for this work shall be certified by the dealer (*See Note 1 opposite*) to comply with the following specifications:

Fine aggregate for this work shall consist of well graded natural or artificial sand, or other approved inert materials with similar characteristics, taken from sources that have furnished satisfactory materials for previous concrete work for several years, or if from a new source shall be thoroughly tested for soundness and permanence. It shall comply with the provisions of the Tentative Purchase Specifications for Concrete Aggregates, E5-A-26T, (or latest standard of the Institute) of the American Concrete Institute. In applying these provisions the sieve analysis of dried aggregate shall show as follows:

Passing No. 4 sieve—not less than 85 per cent.

Passing No. 50 sieve—between 5 and 30 per cent. (*Note 2 opposite*)

Passing No. 100 sieve—not more than 5 per cent.

Fineness Modulus—not less than 2.50

and the weight of clay and silt removed by decantation shall not exceed 3 per cent.

Coarse aggregate for this work shall consist of crushed stone, gravel, air-cooled blast-furnace slag, or other approved inert materials of similar characteristics, taken from sources that have

Note 3

This provision is essential for uniform concrete quality. Various portland cements and even the same brand of cement from different mills may vary in color, strength, rapidity of set, and workability. Changes in the character of grading of the sand, gravel or stone will cause variations (often large) in the workability of the concrete and in the amount of water required to produce a workable concrete on any particular job. Since both the strength and water-tightness of concrete depend on the ratio of water to cement, any changes in materials, resulting as they do in changes in water content, affect the quality and uniformity of the concrete to an important degree. Variation in kind of sand or gravel or stone may also affect the color and texture of the concrete and give a blotchy appearance. The Contractor's orders for materials for this work must include this requirement of a constant source and unchanging characteristics for the entire job.

Note 4

Cement will absorb water rapidly from the ground or even from the air on humid days. Water absorbed in this way hydrates the cement in the sack and destroys or greatly impairs its ability to make concrete. Cement must not be permitted to hydrate until the proper time—when it is placed in the mixer with the water, sand and gravel (or stone).

Note 5

Uniform concrete of specified quality can only be made by accurate measurement of each ingredient and by the continued repetition of this measurement in each successive batch. If the sand and gravel piles are permitted to overlap it is not longer possible to accurately measure either material. With too much sand the concrete becomes too stiff, while if water is added to restore its workability, the strength is reduced. With too much gravel or stone the concrete becomes too wet and tends to separate, the mortar running away from the stone. This results in concrete of uneven quality and also frequently in sand streaks or stone pockets and the extra cost of patching.

Gravel or stone piled up carelessly tends to separate, the coarser particles running down outside to the bottom of the pile.

(*Note 5 Continued Page 74*)

furnished satisfactory materials for previous concrete work for several years or if from a new source shall be thoroughly tested for soundness and permanence. It shall comply with the provisions of the Tentative Purchase Specifications for Concrete Aggregates, E5-A-26T (or latest standard of the Institute) of the American Concrete Institute. In applying these provisions the sieve analysis of dried aggregate shall show as follows:

Passing 1-inch sieve—not less than 95 per cent.

Passing No. 4 sieve—not more than 10 per cent.

Passing No. 8 sieve—not more than 5 per cent.

Fineness Modulus—between 6.30 and 7.50.

The weight of slag shall not be less than 65 lb. per cu. ft. The weight of soft, friable, thin, elongated or laminated pieces shall not exceed 3 per cent.

Aggregates shall be so stored and handled at all times as to remain separate from each other and free of all foreign material until placed in the mixer. (*See Note 5*) Drainage shall be provided where necessary to avoid accumulation of water in aggregates.

Water shall be taken from the usual drinking supply and shall be paid for by this Contractor.

The same materials shall be used throughout the work, including the same brand of Portland Cement from the same mill, the same sand and gravel or crushed rock from the same source and from the same bin at the screening plant. (*See Note 3*) This Contractor shall be fully responsible for the correct certification of all materials by the material dealer, as provided above. In case of doubt this contractor shall pay for the necessary tests to be made under the direction of the Architect or Engineer.

5. Storage and handling of materials on the job

Cement shall be stored in a weather tight shed or building if kept on hand more than three days. If used promptly, it may be stored on a platform raised above the ground or so stored or handled as not to permit it to take up moisture before being placed in the mixer. All the cement in any container, in which part of the cement has become caked or has otherwise deteriorated, shall be entirely removed from the work. (*Note 4*)

Sand and gravel (or stone) shall be kept entirely separate from each other, until placed in the mixer. (*Note 5*) They shall be

(*Note 5 Continued from Page 72*)

When this has occurred the effect is similar to that of mixing the sand and gravel. When all coarse particles are used for gravel, the concrete becomes too wet. When all fine particles are used the concrete becomes too stiff. In either case the uniformity of the concrete is lost, the cost and labor of placing the concrete is increased, and the value of the finished work is impaired. The stock piles should be placed and removed in horizontal layers and all sliding or rolling particles in the piles avoided as far as possible.

Note 6

On the Pacific Coast and in one or two cities elsewhere, structural grade new billet steel may be used, provided the Engineer's approval is secured. Higher stress is permitted on intermediate grade new billet steel or on rail steel than on structural grade new billet steel and this substitution can only be made when the stresses used in the Engineer's design are suitable for structural grade steel. In other parts of the United States, the dealers in reinforcing materials have standardized on the two grades of steel mentioned in article 6 and no other grade is available except on special mill order.

Note 7

No dependence can be placed on the long discredited practice of lifting the bottom steel off the forms, as the concrete is placed, and depending upon the concrete or on pebbles or stones taken from the concrete and placed under the bars, to keep the bars up. Properly designed, rigid supports must be provided and completely placed before concreting is started.

The strength of a reinforced concrete structure, and frequently its fire safety also, depends on the placing of the steel at its designed location as shown on the plans. The load on a chair is frequently very heavy, as when a heavy man carrying one end of a runway steps upon it. Chairs and spacing devices made of soft steel or of too light section, are readily flattened down or bent out of shape. Such devices must not be used, as they only give a false sense of security and the money spent for them is largely wasted.

Note 8

This Contractor should allow in his estimate for a small supply of $\frac{3}{8}$-in. round bars which will be available for filling in

(*Note 8 Continued Page 76*)

stored in bins or dumped only on clean platforms or pavements, and no dirt, rubbish or other foreign material shall be permitted to mix with them. In dumping gravel or stone the various sizes of particles shall not be permitted to separate, but the mass must be kept well graded at all times. (*Note 5*)

6. Reinforcing steel and inserts

All reinforcing steel for this work shall be furnished and placed by this Contractor. All bars shall be of intermediate grade new billet stock or of rail steel, meeting the standard specifications of the American Society for Testing Materials (Serial Designation: A 15-14 or A 16-14). (*Note 6*) All bars shall be supported at the proper height and at the proper spacing as shown by the drawings, using precast concrete or metal chairs, spacers and supports to hold them securely in place. (*Note 7*) Metal chairs, spacers and supports shall be made of stiff springy steel of such strength as not to be flattened out or be displaced by workmen. (*Note 7*) All reinforcement as used shall be thoroughly cleaned of mill or rust scale and of coatings of any kind that will destroy or impair the bond between concrete and steel. All reinforcement shall be carefully formed to the shapes and dimensions shown on plans. Tie wire shall be soft annealed iron wire not smaller than No. 18 gauge.

All bent bars, ties and stirrups shall be tagged with metal or other persistent identification marks to insure their location in the proper part of the structure as shown by the drawings. Heating of reinforcement or rebending of bars will be permitted only when the entire operation is approved by the Architect or Engineer.

All portions of the concrete work shall be provided with reinforcement (unless resting directly on firm, undisturbed ground.) Any gaps in the reinforcement as shown on the plans shall be suitably filled, using not less than $\frac{3}{8}$ in. round bars at 6 in. on centers in one direction in any case. (*Note 8*)

7. Formwork

Forms for this work may be made of wood, steel or other material which has been satisfactorily used on similar work for this purpose. Where panel forms are re-used they shall be thor-

(Note 8 Continued from Page 74)

gaps as specified. By the term "gap" is meant a *small* unreinforced area such as sometimes occurs between pipe sleeves, or where the regular groups of parallel bars leave a small triangle between them. Any unreinforced space above 5 sq. ft. in area, or extending for any considerable distance should be promptly called to the attention of the Architect or Engineer and a request made for instructions.

Note 9

Wooden formwork tends to dry out and to open up cracks between the boards. Dirt gathers in these cracks and when the forms are wetted, either by rain or in preparation for concreting, the dirt prevents the cracks from closing and as a result, the surface of the finished concrete shows a succession of unsightly ridges where the mortar has filled these open cracks. The swelling of the forms also causes movements at the sides of the building, at elevator and stair openings, etc., and requires expensive cutting to plumb up the walls and shafts. This cutting may reduce the fireproofing or even expose the steel to rusting. All this can be easily and inexpensively prevented by the simple expedient of keeping the formwork always damp, as specified.

Note 10

It is much better to cut a hole of the size of the end of a 4x4 or a 6x6 and build in a shore in advance so as to bear directly on the concrete, than it is to remove the forms and then "re-shore" (put in new posts), wedging the same against the new slab. Re-shoring operations permit the slab to sag and then force it back again to an undetermined amount. Slabs have frequently been weakened or even cracked by re-shoring and this practice should be entirely discontinued. Built-in shores must be selected so as to have sufficient stiffness to support their load after the bracing has been removed with the regular formwork.

oughly cleaned and repaired to produce a true, even surface. All forms shall be mortar-tight and sufficiently rigid to prevent sagging between supports. Surfaces of metal forms shall be free of marked irregularities, dents or sags. Knotholes or broken places in wood forms shall be covered with metal patches. All formwork shall be so designed (cleanout openings at base of columns, at bottoms of deep girders, etc.) as to permit of ready cleaning and inspection after the reinforcement has been placed. For spans in the structure in excess of twenty feet, the bottoms of beams, or girders, shall be cambered, allowing one-twenty-fourth of an inch rise per foot of span. Joints in formwork shall be either horizontal or vertical. All interior angles not occurring on an exposed face of the structure shall be formed with a one-inch bevelled fillet, unless otherwise called for on plans.

Wooden formwork shall preferably be kept continually moist after being erected, so as to prevent the opening up of cracks in which dirt may lodge. (*Note 9*) Paraffin or other suitable formwork oil shall be applied to forms immediately before the reinforcement is placed. In no case shall the oil be permitted to coat the reinforcement. No oil shall be used where the concrete surface in contact with the formwork is later to receive plaster. Concrete shall be placed as soon as possible after the reinforcement has been placed and inspected, so as to prevent the collection of dust and dirt on the oiled formwork.

The design of the formwork shall include all necessary shores built in place in advance of concreting in cases where portions of the forms require to be removed before the concrete on them is ready to support itself. These shores shall not be disturbed in the removal of the adjacent formwork. No re-shoring will be permitted. (*Note 10*) Formwork supports shall be trussed as necessary for stability and to prevent settlement when foundation conditions for shores are bad. Where internal ties are necessary bolts or rods shall be used; they shall be so arranged that no metal shall be left within one inch of the surface of the concrete. All holes left by the withdrawal of these bolts or rods shall be neatly patched. Tie wires will be permitted only for concrete members entirely protected from the weather (or other corrosive exposures) and only by especial permission of the Architect or Engineer.

Note 11

The rapidity with which concrete gains in strength depends on several factors, which must be considered in connection with removing form work. Certain portland cements give a higher strength at early ages than others. Concrete gains strength much more rapidly at high temperatures than at low temperatures. Stiff concrete gains strength more rapidly than sloppy concrete. Concrete must have moisture in order to maintain its increase in strength, and concrete from which moisture has been rapidly evaporated within a few days after placing may be expected to be weak and leaky. On any particular job the temperature since the concrete was placed is most important, while the other items mentioned are important in case any doubt exists as to the wisdom of removing form work.

Note 12

Accurate measurement, providing the same amount of each material in every batch, will result in concrete of the same workability and will make all the operations of mixing, transporting, and placing concrete, as simple as possible. Inaccuracy of measurement will cause variation in the consistency of the concrete with corresponding uncertainties in the mixing, transporting, and placing and will result in concrete of unequal strength, water-tightness and quality. Stoppages in the chutes, with their attendant mess and expense, are commonly due to variations in measurements (or in grading) that result in a too-stiff batch that sticks, or a too-wet batch that races. All such troubles may be readily avoided by careful control of the materials (see note 5, page 72) and consistent accuracy of measurement.

An experienced workman shall be employed at all times during the placing of concrete to watch the formwork and its supports and to strengthen any portions that may show signs of sagging, displacement or failure.

8. Removing formwork

This Contractor shall be solely responsible for the safety of the construction during and after form removal, and no act of the Architect or Engineer or of his representatives on the work shall relieve this Contractor of such responsibility. Subject to this limitation, and measured in days of good curing temperatures (not less than 60° F., twenty-four hour average), forms for columns shall not be removed in less than one day; forms for slabs or other members whose span does not exceed five feet nor eight times their depth and which carry only their own weight (or in which all super-imposed loads are adequately supported by built-in shores [*Note 10*]), shall not be removed in less than three days. Other forms shall not be removed in less than seven days. At average temperatures, lower than 60° F., a longer period shall elapse sufficient to produce the same strength of concrete. (*Note 11*)

9. Patch formwork

Forms for patches (where temporary openings have been left for any reason) shall be rigidly supported, being posted from the floor below or suspended by rods or bolts and not hung by twisted wires or other means that will permit displacement of the patch formwork, and unsightly surfaces.

10. Measurement of the batch

Job measurements shall be so made as to insure that each successive batch of concrete is made up of identical volumes (or weights) of cement, water, sand and gravel (or stone). (*Note 12*) Each batch shall be proportioned to use only whole sacks of cement, unless a suitable container is provided to measure accurately the required amount of cement from broken sacks, or unless the required amount of cement is determining by weighing on the job. Sand and gravel or stone may be measured in measuring boxes containing the required volume for one batch, such boxes being struck off level full at each use, or any equivalent accurate method may be used. At least one wooden box

Note 12A

In this specification the *water-cement ratio* is the fixed requirement. The strengths given are based on the usual experience with average materials. The Architect or Engineer, under this specification, does not require this strength, but he does reserve the right to require the use of other common and readily available materials, in place of those selected by the Contractor, should the latter prove to be exceptional and to result in lower strength than that intended. The approximate mixes are likewise not binding upon the Contractor, who is free to use as much aggregate as he can while maintaining the specified water-cement ratio and a workable consistency.

of one cubic foot capacity, with handles, shall be provided for checking volume measurements. Unless weighed or measured in an accurate measuring tank, water shall be measured in a pail with a line painted on it to give the proper water level at each filling.

The proportions of cement, water and aggregate (sand and gravel) shall be taken as follows:

Class A concrete, requiring a strength of approximately 2000 lb. per sq. in. at 28 days, shall be made with 7.5 gallons of water per sack of cement. The approximate ratio of the volume of the cement to the sum of the combined volumes (measured separately) of the sand and gravel (or stone) will be 1 to 5.75 (*Note 12A*) Unless otherwise noted on the plans, all concrete will be Class A concrete.

Class B concrete, requiring a strength of approximately 2500 lb. per sq. in. at 28 days, shall be made with 6.75 gallons of water per sack of cement. The approximate ratio of the volume of the cement to the sum of the volumes of the sand and gravel (or stone) will be 1 to 5. (*Note 12A*)

Class C concrete, requiring a strength of approximately 3000 lb. per sq. in. at 28 days, shall be made with 6 gallons of water per sack of cement. The approximate ratio of the volume of the cement to the sum of the volumes of the sand and gravel (or stone) will be 1 to 4.25. (*Note 12A*)

Equivalent weights may be used in place of volume measurements for all classes.

In determining the amount of water, allowance shall be made for the free (surface) water on the aggregate. In the absence of job tests, sand shall be considered to carry 0.25 gallons of surface water per cu. ft. if damp, and 0.8 gallons per cu. ft. if wet. Gravel or stone shall be considered to carry 0.15 gallons per cu. ft. if damp, and 0.5 gallons per cu. ft. if wet.

Trial batches shall be made on the job to determine the final volume (or weight) of sand and gravel (or stone) to be used. The water-cement ratio shall not be changed but the amount of aggregate shall be increased or decreased to give the proper workability of the concrete as specified in article 11. The division of the aggregate between sand and gravel (or stone) shall be

Note 13

Sand and gravel (or stone) may be considered as fillers in the concrete, the quality of the concrete depending primarily on the water-cement ratio, which determines the quality of the "paste." Variation in this proportion of sand to gravel (or stone) affects the workability of the concrete greatly and the trial batches should cover a considerable range so that the best proportion for the economical transporting and placing of the concrete may be selected. The limiting proportions given here are determined from experience with a very wide range of materials.

determined by trial so as to give the most workable and non-separating concrete, but the volume of sand shall not be less than one-third nor more than one-half of the total volume of the sand and gravel (or stone) measured separately. (*Note 13*)

11. Mixing concrete

Concrete shall preferably be mixed in a power driven rotary batch mixer. In no case shall the capacity of the mixer be less than a one sack batch and a large mixer with a capacity of two or three sacks of cement (with the corresponding quantities of water, sand and gravel (or stone) shall be provided except on very small jobs. Other devices, mixing concrete equally well, may be used with the consent of the Architect or Engineer. The capacity of the mixing equipment shall be such as to produce all the concrete required at any one level above the foundations in a single working day. In case of very large areas to be built at a single level this requirement may be waived by the Architect or Engineer.

Mixing equipment shall be cleaned at frequent intervals so as to keep all parts operating efficiently. Batch mixers shall be fully discharged before the next batch is admitted to the drum. Concrete which has partially hardened shall not be remixed with added water. Such concrete shall be wasted.

The requirements of Article 10 apply equally to hand mixing when this is resorted to for small quantities of concrete. Hand mixing shall be done in a clean, watertight, mortar box or on a level, watertight, clean wood, metal of concrete floor or platform which has been thoroughly wetted to prevent absorption of water from the concrete. The sand shall first be spread in a layer of uniform thickness on the mixing platform. The gravel or stone shall then be placed on top of the sand in a second layer of uniform thickness and covering the entire first layer. The two layers shall then be turned and mixed with shovels for at least three complete turns and until the mixture is uniform in color and appearance. The mixture shall then be levelled off and the cement spread in another layer of uniform thickness over the entire mass. This shall then be turned with shovels for at least three additional complete turns and until the mixture is uniform in color. The material shall then be levelled off, depressions

Note 14

Hand mixing of concrete is generally prohibited, not because a satisfactory concrete cannot be made by this method, but because the work is often done in a careless and lazy manner. This Contractor and his superintendent on the job must give personal attention to hand mixing operations, training a crew of men to do the work systematically and thoroughly and affording them time to carry out these specifications properly. Each "turn" must handle *all* of the material and there must be a clear gap in the pile (up to the time that water is added) between the portion turned and that not yet turned on each operation. Thus the pile moves bodily to the left at one turn and back to its original position at the right on the next turn, etc. The platform or mixing box must be of sufficient size to permit of this movement for a full batch. Much skill is shown by some workmen in securing a thorough mixing of the materials, by the proper twist of the shovel, and also much skill in working the water into the mixed cement, sand and gravel (or stone) evenly, and without permitting any to escape. Workmen showing such skill must be employed at this work in preference to those without it.

Note 15

The greatest economy, resulting from the use of more sand and gravel (or stone) with the same amount of cement and water, will be found with the stiffer consistencies. Too stiff a consistency will increase the difficulties of placing, however, unless the concrete can be tamped in placing. In the case of footings or of floors laid on the ground, very stiff concrete, consolidated by tamping, is permissible and economical. The sloping portions of footings may be built without forms in this way. Since the water-cement ratio is definitely fixed, and the consistency limited by these specifications, the greater amounts of aggregate necessarily involve stiffer concrete.

The consistency of concrete, made from any fixed set of materials, may be judged by the slump test, using a truncated cone-shaped mold as described in the A. S. T. M. specification D 138-2-T. This device is especially useful in case the moisture in the aggregate varies rapidly (as after rainy weather), to de-

(*Note 15 Continued Page 86*)

shall be made in the surface uniformly distributed and of sufficient size to hold all the mixing water for the entire batch and this water shall be poured into the depressions to give as near uniform distribution as possible. With the shovel, work the ridges of mixed cement and aggregate so as to take up all the water without letting any escape, and then turn the mass with shovels for at least two additional complete turns and until it is uniform in consistency. (*Note 14*) Only enough water shall be used to give a quaking, jelly-like mass, from which the water will not separate and run out on the platform, after mixing is completed.

The consistency of the concrete when mixing is completed, shall be such that the concrete flows slowly with a rounded edge, and without any visible separation of the water or of the mortar from the gravel or stone. For concrete to be placed in thin walls or about congested reinforcement or in other difficult places, the rounded edge may be permitted to be quite thin, while for concrete placed on a level deck with very little steel the concrete should be as stiff as possible while still showing a decided tendency to flow. (*Note 15*)

12. Time of mixing

One full minute shall elapse between the time when all the material for any given batch is placed in the mixer and the time when the first concrete of that batch is discharged from the mixer, except that a portion of the mixing water may be introduced during the first ten seconds of the mixing time.

13. Conveying concrete

The consistency of the concrete at the completion of mixing, as specified above, shall be maintained during the transportation of the concrete from the mixer to the forms. The methods used by this Contractor on this work shall be such as to comply fully with this requirement. (*Note 16*) Separation of the mortar from the coarse aggregate shall not be permitted at any stage.

14. Placing concrete

Before commencing the placing of concrete all formwork shall have been thoroughly washed (*Note 17*) and shall be clean and free from construction dirt and debris, all reinforcement shall be properly placed and supported and all inserts and openings shall be completely installed for a distance of at least thirty feet ahead

(*Note 15 Continued from Page 84*)

termine quickly and accurately the change in mixing water required to keep the specified consistency.

Note 16

Chutes may be used in transporting concrete provided the results on this work at all times meet the requirements of article 13. It is possible to use chutes so that this is done, but it is all too common that failure results in the attempt. Chutes must be sloped at just the right angle to cause the concrete to flow without separation. A hopper must be provided at the end of the chute and the concrete wheeled or shovelled from this hopper into the forms; the only permissible exception (permitting concreting direct from the chute) being where concrete is placed in a large mass, as in a footing, where there is no chance for it to flow out to a thin edge and to separate. Carts or wheelbarrows will be used more commonly on jobs of the small size covered by this specification.

Note 17

Even in freezing weather the formwork should be cleaned with water or steam, as this is the only practical means of removing the loose dirt. If the temperature is sufficiently high for the proper placing of concrete under this specification, the water will not freeze.

Note 18

On small jobs it is even easier than on large ones to change frequently from concrete to "grout" and so keep the mortar always fresh and soft when the concrete is placed. In hot, dry weather, especially, good judgment must be used in the timing of mortar batches, as in such weather the thin edges of the mortar dry quickly and there is a tendency for such dried edges to scale off later. For this reason the consistency of the mortar should always be such that it flows with a slightly rounded edge. The amount of water to be used to give this consistency is readily determined by trial.

Where the steel is very congested, as in thin walls, so that the concrete is separated by hitting rods repeatedly as it falls, this difficulty may be readily overcome by using a portable metal

(*Note 18 Continued Page 88*)

of the concreting operation on all sides. Runways shall have been placed and the resulting dirt cleaned up and any steel disturbed shall be properly replaced. Column capital forms or other depressions at which the washing water may gather shall be drained and the concrete shall not be deposited in standing water. In weather requiring heating of the fresh concrete in place, the salamanders or other heating devices shall be in operation before concreting starts. No snow or ice shall be left in any part of the forms.

In columns, or similar slender vertical or slightly inclined members, the bottom shall be filled with mortar (concrete from which the coarse aggregate and much of the usual water has been omitted) which shall be sufficient in quantity and proper in consistency to work up with the concrete and to fill all spaces between closely spaced bars or between the steel and the formwork. In girders or other similar members, in which the congestion of steel makes placing difficult, the lower part of the forms shall be filled with mortar in the same manner. This mortar, in either case, shall be placed immediately in advance of placing the concrete and shall not be permitted to dry out or set so as to lose its mobility before the concrete is placed on top of it.

Concrete shall not be dropped through reinforcing steel (as in walls or beams with top reinforcement) so as to cause separation of the coarse aggregate from the mortar on account of repeatedly hitting rods as it falls. In such cases hoppers, and, if necessary, vertical ducts shall be used in the wall forms or on the outer face of the wall forms, (*Note 18*) or other means employed so that the concrete may reach the place of final deposit without separation. Concrete shall be deposited in small amounts at many points and shall not be required or permitted to flow along the forms for more than a very short distance, or from beam forms into column or wall forms.

Concrete shall be worked around the reinforcing rods, inserts, etc., and against vertical forms as it is being placed, so as to produce smooth, dense, surfaces and complete filling of all spaces within the forms but care shall be taken not to over-spade the concrete so as to cause segregation locally. Concrete being placed with a very stiff consistency (as in paving base or in sloped foot-

(*Note 18 Continued from Page 86*)

duct with a hopper top that may be inserted inside the form so as to conduct the concrete past the bars, or when the space between bars is very small, wooden ducts may be built by housing in the space between adjacent studs and cutting openings through the side form to admit the concrete below the steel. A duct 3 in. to 4 in. wide by 16 in. to 20 in. long in cross section is adequate for this purpose. In the case of the outside wooden duct a baffle fastened to a long handle should be lowered inside the forms and placed directly opposite the opening in the form and close up to the form to prevent the concrete from segregating as it flows through the hole.

ings concreted without forms) shall be consolidated by tamping or vibration of the concrete or by vibration of the forms (never by vibration applied directly to the reinforcement).

15. Construction joints

All the concrete in vertical members (such as columns or walls) shall have been placed at least two hours before any concrete is placed in the beams or slabs directly over such columns or walls. This interval may be increased if desired (over night, for example) but necessary precautions shall be taken to insure that the new concrete bonds to the older concrete, if the interval exceed thirty hours, or if the weather is hot and drying. In any case, any excess water or fines that may rise to the top at a horizontal construction joint, shall be removed and the concrete cut away as necessary to insure strong, dense concrete at the joint.

Construction joints in beams and slabs shall be vertical and shall be located close to the center of the span, unless a beam intersects a girder at this point, in which case the joint in the girder shall be offset a distance equal to twice the width of the beam and inclined reinforcement provided to take the shear across the offset construction joint. Construction joints shall be made with a key at the center of depth and with reinforcement near each face of the member, additional bars, at least 60 bar diameters in length, being provided where steel is lacking in either face of the member cut by the construction joint. The added bars shall have a cross sectional area not less than .0025 of the area of the member cut by the construction joint. The member of construction joints in beams and slabs shall be kept to a minimum. (See article 11—capacity of mixer required.)

16. Placing against hardened concrete

Before depositing new concrete on or against concrete which has set, the forms shall be retightened, the surface of the set concrete shall be roughened as required by the Architect or Engineer, thoroughly cleaned of foreign matter and laitance, and saturated with water. The new concrete placed in contact with hardened or partially hardened concrete, shall contain an excess of mortar to insure bond, and shall not be placed until the saturated surface has become dull (but not lightened in color) through absorption or removal of all excess water.

Note 19

Watertight concrete can readily be obtained without the use of any admixture or compound. The concrete must be so proportioned as to have sufficient mortar to fill all the voids in the gravel (or stone), which is also a requirement for good workability and easy placing. If, then, the concrete is held to the consistency specified above and placed with care to prevent segregation (stone pockets or sand streaks indicate bad segregation and careless work) the concrete will be watertight when the cement is properly hydrated. It is the experience of investigators that almost all usual concrete mixes produce waterproof concrete under laboratory conditions. The difficulty in the field probably arises largely from segregation during transporting and placing of the concrete and from joints where the concrete is not made to bond properly. (See article 16.) The process of hydration requires favorable conditions—warmth and moisture always present—and a reasonable length of time. The strength as well as the water-tightness is greatly improved by long, warm, moist curing. Warmth without moisture simply dries out the concrete and all gain in strength or water-tightness ceases in dry concrete. Moisture without warmth is equally useless or may be harmful if freezing occurs. Below 40°F concrete is practically dormant. It is important also that concrete is provided with both warmth and moisture when it is young, as the chemical activity decreases with age.

Note 20

Concrete deposited in a trench or on the ground can be "effectively enclosed" by a covering of hay on all exposed surfaces or by laying a tarpaulin over the trench and covering it with hay. Concrete in large masses develops ample chemical heat for its own curing, provided this heat is prevented from escaping. In elevated work, the problem of effective enclosure is more difficult, and care must be taken with the usual enclosure of tarpaulins (heated with salamanders) to properly support the tarpaulins with timber to prevent openings in windy weather and to keep the tarpaulins away from the surface of the concrete at edges of a floor. The arrangement must be such as to provide warm air surrounding

(*Note 20 Continued Page 92*)

17. Defective concrete

Any defective work discovered after the forms have been removed, which in the opinion of the Architect or Engineer cannot be repaired satisfactorily, shall be removed at once and replaced. No compensation will be allowed for patches or replacements because of defects. Defects shall include honeycombing, or other surface blemishes; or portions which through lack of uniformity in the concrete render them, in the opinion of the Architect or Engineer, inadequate in strength; or improper alignment of outside or exposed faces.

Patching where permitted shall be done only under the supervision of the Architect or Engineer. The cavity to receive the patch shall be wetted thoroughly and allowed to stand sufficient time for the surface of the cavity to become dull but not lightened in color. The patching material shall be of the same material and in the same proportions as the concrete receiving the patch. It shall be worked into place in a dry consistency and shall be thoroughly rammed. The patch shall be built up and supported in such a manner as will prevent sagging. Especial care shall be taken to prevent patch concrete from drying out and to cure it thoroughly, as required in article 18 below.

18. Curing of concrete

Concrete, after taking its initial set, shall be protected from drying out at all times during the first seven days after it is deposited. If the concrete is so located as to require especial water tightness this curing period shall be increased to twenty-one days. (*Note 19*) Protection may be afforded by replenishing the supply of moisture to offset losses by evaporation or by sealing in the water carried by the concrete so that it cannot later evaporate. All portions of the structure shall be kept moist and warm simultaneously for the full curing period of seven or twenty-one days and any portion of the time during which either moisture or warmth is lacking shall not be counted as effective for curing.

19. Cold weather protection

Artificial heat shall be provided and effective enclosure maintained, whenever necessary, to insure that the air in contact with concrete, for a period of seven days after placing, shall not fall below 50° F. for any considerable time. The temperature within

(*Note 20 Continued from Page 90*)

all the concrete. Where, in moderate weather, only the story below the freshly placed concrete floor is enclosed, the upper surface of the concrete must be protected with sawdust, hay or similar insulating material. Temperatures in excess of 100°F in the enclosure, tend to dry out the concrete. If salamanders with water-evaporating pans over them are used, or if live steam is used for curing, there is no danger (and in fact much gain) from using moderately high temperatures in the enclosure.

Note 21

In heating sand or gravel (or stone) on a small job by the usual method—maintaining a fire in a metal tunnel through the storage pile—considerable moisture will be driven off and the mixing water may be increased to maintain the usual warm weather consistency (best measured as slump—see note 13, page 82) for the water-cement ratio specified. Where live steam is permitted to escape in the piles, on the other hand, much water is introduced into the sand and gravel (or stone) and the measured water must be *reduced* to maintain the consistency. Heating the water alone is the cheapest way to heat concrete and introduces the fewest complications, and there is no limit to the temperature of the water, provided it is introduced into the mixer with the sand and gravel (or stone) and so cooled to 150°F or less before it meets the cement. Very hot water may cause a flash set of the cement and should not be used with hand mixing. A properly operated kerosene torch in the mixer is effective, provided sufficient time is allowed to heat the entire mass of the concrete.

Note 22

Since the strength, water-tightness and other desirable properties of concrete are dependent on a low water-cement ratio, any increase in the water content by rain or snow or ground water has the same effect as increasing the mixing water. Where concreting must be continued in bad weather, the consistency should be made as stiff as can be handled properly (with extra spading) and the extra cement added as specified. As soon as possible the fresh concrete should be covered. In many cases, it is possible to

(*Note 22 Continued Page 94*)

the enclosures shall not exceed 100° F. (*Note 20*) Moisture shall be provided within the enclosure as required by article 15.

When necessary to insure that the fresh concrete shall reach the forms at a temeprature of not less than 50° F., the materials (water only in moderately cold weather and aggregates also in very severe weather) shall be heated. The concrete may be heated in the mixer by any means that will not impair its quality, (*Note 21*) and subject to the approval of the Architect or Engineer. Dependence shall not be placed on salt or other chemicals to prevent concrete from freezing. No frozen materials or material containing lumps of snow or ice shall be used. Manure when used for protection, or moisture from manure, shall not be applied directly to, nor be permitted to come in contact with concrete.

20. Water supply

An adequate supply of water for cleaning formwork and for curing concrete shall be installed so as to be available (at all levels) before concreting starts and for at least the period of curing as required by article 15.

21. Protection of concrete work

Concrete shall be protected from washing and from the addition of excess water in the concrete during stormy weather, until such time as it has sufficiently hardened to resist washing or penetration. One extra sack of cement, per cu. yd. of concrete mixed during stormy weather, shall be added to maintain the required water-cement ratio and surface water shall be drained away from the fresh concrete during heavy rains. (*Note 22*) Additional measures shall be taken to safeguard the quality of the concrete, if necessary.

Concrete work shall be protected from injury during the removal of formwork and from injury due to the storage or movement of materials in or over the structure during construction.

22. Cement floor finish and under-finish

Cement floor finish or under-finish (cement finish shown on plans to be covered by linoleum or other surfacing material), if less than one and one-half inches thick, shall be placed as soon as the structural concrete has hardened sufficiently for workmen

(*Note 22 Continued from Page 92*)

leave the finished surface slightly low and to bring it up to grade by scattering on a mixture of dry sand and cement (in the same proportions as the sand and cement in the regular mix) to absorb the extra water. In footings it is permissable to use a dry batch of concrete for the same purpose. All methods used for this purpose must be approved by the Architect or Engineer.

Note 23

A steel trowel is a dangerous tool and may easily be (generally is) overworked. Dusting of concrete floors is commonly due to overworking with a steel trowel and to the use of fine sand. For work exposed to the weather, such as sidewalk finish, it is much better not to use the steel trowel at all. A wood float will give a sufficiently smooth and far more permanent finish. Where a trowelled finish is required the finishers must be restrained from starting until a single trowelling will leave the surface completed. Each additional trowelling damages the surface by bringing additional fine material to the top. Water should never be sprinkled on a finish to permit of easier trowelling—it is certain to result in dusting and impermanence. Floor finish should have especially good curing, as they are subject to severe wear.

Note 24

A mortar (or concrete) topping for concrete slabs, far superior to the usual product, may be secured at a very reasonable cost in spite of its novelty to the average cement finisher, by the following method. Instead of the usual mortar, use for topping a *concrete* composed of one part cement, one part coarse grained sand (not over 10 per cent passing the No. 50 sieve) and one and one-half to two parts of gravel or crushed stone retained on the No. 4 sieve and passing the $\frac{3}{8}$-in. screen. Mix this with 4.5 to 5 gal. of water per sack of cement for a period of two minutes (to give a fat, easily worked concrete). Place, screed and wood float this topping, then allow to stand for from 30 to 48 minutes, so that it becomes sufficiently stiff to prevent the fine material from being drawn to the surface by the steel trowel. Finally go over with the steel trowel, to give a dense, hard surface. With this kind of

(*Note 24 Continued Page 96*)

to walk on it, or a dry mixture of *coarse* sand (*Note 2*) and cement, in the same proportions as the mortar in the concrete, may be distributed uniformly over the surface of the concrete before initial set has occurred. In either case the method used must result in the formation of a mortar surface layer of uniform consistency (about four-inch slump) without excess water which shall be darbied to a true surface and trowelled later to a dense, smooth finish. The surface shall not be gone over more than twice with a steel trowel. For surface exposed to the weather, a wood float shall be used, entirely replacing the steel trowel. (*Note 23*)

Stair finish shall be laid in the same manner as floor finish.

Mortar for floor finish or under-finish shall be made with the same proportions of cement and sand as is used in the concrete to which the finish is to be applied. In no case shall the volume of cement exceed 40 per cent of the volume of the fine aggregate. (*Note 24*)

23. Concrete floors on ground

Concrete floors shall not be placed directly on the ground, other than sand or similar material, except for the lower course of two-course floors having a water-proofing layer between the courses. One-course concrete floors on ground, sidewalks and parking strips, shall be laid on top of a plain cinder, stone or sand filling at least five inches thick, tamped and finished accurately to the required grade of the underside of the concrete. They shall be of Class A concrete of the thickness shown on plans (but not less than five inches thick.) They shall be finished as specified in general for floors or as called for on plans.

Open joints ($\frac{7}{8}$ inch wide) shall be left between such a floor slab and the enclosing walls and columns and this joint shall later be caulked with asphalt mastic. In case the floor slab is exposed to more than 40° F. variation in temperature, or is subject to changes in moisture content, or is more than 100 feet long in any direction, similar joints shall be made at intervals of not to exceed one hundred feet and caulked in the same manner.

(Readers are referred to the Journal *for January, 1931, for discussion which may develop. Such discussion should reach the Secretary by December 1, 1930.)*

(Note 24 Continued from Page 94)

topping (with very little fine material in it), after it has been allowed to stiffen, the damage from steel trowelling is avoided and fine, dust-free, weather-resistant surfaces are produced.

A topping of concrete instead of the usual mortar is not required by this specification. It is *permitted* as an alternate, unless the Architect or Engineer definitely calls for it, in his typewritten specification.

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Discussion of Report of Committee 502

Construction Specifications for Concrete Work on Ordinary Buildings (502-30T)

The discussion of this specification is continued from the Journal *for March* (A. C. I. Proc., *Vol. 26, p. 580*). *Written by Mr. Lord, as Author-chairman of Committee 502, with the cooperation of the critic members, it was published in the* Journal *for November, 1929* (*Vol. 26, p. 1*) *and presented to the 26th annual convention. As there amended* (*See Vol. 26, p. 580*) *it was given tentative adoption. If unamended in substance it will be in order at the 1931 convention to move its submission to letter ballot as a Standard of the Institute.*— Editor

BY GEORGE A. SMITH*

I am most heartily in accord with the Institute's present plan of standardizing practices involving the use of portland cement, and feel that what is done will be for the best interest of all. With present practice in the manufacture of concrete so variable, I am impressed with the value of such a Specification as that presented by Committee 502. It surely will help greatly in unifying practice.

My attention was directed particularly to Article 10. "Admixtures and Special Cements." If paragraph one were included in a specification and then interpreted literally, such materials could have no part in the program until such a time as it would be possible to obtain all the information stipulated. Any specification should be reasonable and just.

Any material used in concrete should be studied as carefully and thoroughly as time and conditions permit. However, to

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require that materials shall be "thoroughly tested by reputable independent investigators, so as to demonstrate their effect on the strength, elastic properties, permeability and permanence of concrete produced from materials such as are being used on this work," before it may be used appears to be unnecessary in view of the Architect's and Engineer's present knowledge of admixtures and their functioning.

How many tests are made to determine the elastic properties, and permeability or the permanence of concrete made with the particular brand of cement proposed for a particular job? Because portland cement is to be used is no reason why all the properties of the concrete will be the same regardless of the brand used. Different brands of portland cement may have different properties due to different methods of manufacture, to small variations in the chemical composition or to inherent properties of the raw materials. All portland cements are acceptable for concrete work when they pass the minimum requirements of the A. S. T. M. because experience has shown that such cements have usually given satisfactory results when properly used. When this is taken into consideration why should such a question be raised about admixtures which are manufactured under definite specifications and which have given successful service in the past? Elaborate studies for determining the various properties are generally not required because of the time and expense involved in securing the data. In exceptional cases, however, most complete information may be desirable and highly important but such is not the rule especially on "Ordinary Buildings."

Progress in the art of concrete or in any other art is not attained by sitting back and waiting until some independent investigator sees fit to take up the study of some particular product. To insure progress and the introduction of new process and new products, manufacturers are continually studying and establishing definite knowledge regarding the uses of their materials. This information is of unquestioned value to industry and is generally accepted by engineers.

Among the admixtures now on the market many have unknown and unproven characteristics, and the data available are not so extensive as might be desired. There are, however, other admix-

tures which have proven their worth and which have been used successfully for a number of years.

Many prominent engineers and architects have found from their own experience and from that of others that some admixtures are effective in producing certain desirable results and go so far as to incorporate their use in their specifications. What better data are to be desired than the experience of engineers and architects? Is not such information reliable? The proposed specification at present excludes such materials and prevents their being used until such a time as elaborate and comprehensive tests are made under a multitude of conditions.

In order, therefore, that the proposed specification may be consistent in this regard with present practice I would suggest that paragraph one Article 10 be struck out and that the following paragraph be substituted:

Materials to be added other than the usual ingredients of concrete (portland cement, fine and coarse aggregate and water) shall be proven suitable to the satisfaction of the Architect or Engineer. Approval for the use of such additional materials shall be in writing.

Aside from the question of approval or disapproval of admixtures, I am impressed by the incompatability of the proposed specification with the ability, and the rights and prerogatives of the Architect or Engineer. In Article 9 "Constant Source of Supply," we find "having determined upon the source and kind of cement and aggregate to the satisfaction of the Architect or Engineer, etc." Here, from data available, be it from experience or from test, he is allowed to exercise his ability and his judgment in selecting what materials shall be used. Yet, when considering other possible ingredients, we find that the specification is dictating to the responsible head what he shall do and how he shall do it. Is the specification written to govern the Architect and the Engineer or is it to govern the job and the Contractor? If it is the intent, to regulate and control the acts and decisions of the Engineer or Architect, there can be little exception to the specification. However, I do feel that such is not the meaning.

Responsible architects and engineers are capable of judging what materials shall be used and would, in all probability, incorporate as a part of their specification the above proposed amendment. To add the last part of paragraph 1 Article 10 stipulating

what the Engineer or Architect may consider would be a reflection on his ability. As now proposed we have the approval in writing and then some one comes along and dictates "and then only such materials shall be added, etc." Why not let the matter rest upon "to the satisfaction of the Architect or Engineer"?

BY G. W. HUTCHINSON*

Among other details of the material specifications, the manner of presentation of paragraph 10 "Admixtures and Special Cements" is open to question, especially when the first impression is reinforced by the tone of the author's criticism.

In a specification, drawn for national use, the amount of consideration accorded it depends not only upon its practicability, but also upon its interpretation as regards prejudice and commercialism.

It occurs to me that this paragraph should either be eliminated or relieved of the impression it will convey as now worded.

There are admixtures and special cements of several classes. Without doubt, some are not suited for all purposes, and possibly some are unsuited for any purpose. On the other hand, a prejudice is set up against those that have been used with beneficial effect.

It is felt that it is a matter of competition among producers to show the superiority of their product, etc., rather than to give them all a technical "black eye."

Due to the activity of both the proponents and opponents of admixtures, the subject has become prominent during the past few years. Upon analysis, the majority of discussion has been confined to the producers of admixtures and producers of portland cements. The controversy has reached a ridiculous stage in some instances, and reflected no credit to either side. Further, technical magazines and societies have published the "pros and cons" without censorship.

Engineers and architects, who are the judges of admixtures on their individual projects, will be handicapped by the adoption of a specification of this nature. It is assumed that the general

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specification will be promoted, in one way or another, into the building codes of the various cities. Under these circumstances, the architect or engineer for private work, regardless of his desires, finds himself unable to specify an admixture, as his specifications must conform to local building code requirements. Paragraph 10, in question, requires more information about admixtures than we know about the cement and aggregates which appear to be accepted without question.

Without going further than to discuss this paragraph generally, it would appear to the writer that an amendment should be made to remove the present reference and substitute one along the following lines.

> Admixtures and Special Cements, unless specified, shall be used only upon approval of the Architect or Engineer in writing. The producer of any admixture or special cement shall have available, not only complete information regarding his product, but shall also have sufficient test data, from various acceptable sources, to indicate a consistent agreement between them with regard to the effect of the product upon the desired qualities of concrete.

It is well recognized that many of the tests regarding certain admixtures cannot be duplicated at will, and are the result of either poor testing routine, or are "freak" results so often encountered. For this reason, a reasonable agreement between laboratories would be desirable to give the user protection against the exploitation of favorable tests and the discarding of those of unfavorable nature. It would relieve the "salesman" objection, a point well taken by the author in his own criticism.

As stated before, it is desired to relieve this point rather than to attempt to discuss details in connection with the whole specification at this time. It is to the best interests of the Institute to further the use of a specification agreeable to all and at the same time protect the user and secure the most economical concrete.

The paragraph regarding admixtures and special cements is undoubtedly objectionable to a large number, and, on account of the relations between the architect or engineer and the local building inspection departments, is a serious handicap.

Unless the whole specification is considered seriously; accepted in a national sense, and not subjected to criticism during its promotion, it will reflect against rather than to the credit of the Institute.

BY C. E. ELLSWORTH*

Due to the public's confident acceptance of the Institute's work, the Institute has a most serious responsibility to the public.

If Article 10 of the "Construction Specification for Concrete Work on Ordinary Buildings" is permitted to stand, it will virtually exclude all admixtures from concrete made under this Specification.

Will it best serve the public interest to exclude *all* admixtures from concrete manufactured under this Specification?

Of late years engineers have appreciated more and more the fact that impermeability is of even more importance than strength and rigidity, in its effect on the permanence of concrete structures. In fact, never in the history of concrete construction has there been a time when the subject of water-tightness received as much thought and attention as it does at present. And the problem is not confined to such structures as dams, reservoirs, aqueducts, water-treating tanks and basins, where loss of water through percolation is involved, but includes roads, culverts, bridges, piers and abutments, retaining walls and revetments, tunnels, sewers, footings and foundations, basement and building walls, etc., as well.

Concrete surfaces exposed to the weather have a tendency to check and craze; this weathering action being due to alternate and unequal expansion and contraction of the surface concrete, under the influence of varying temperature and moisture content. Water enters the fine cracks thus formed; and in cold weather freezes, enlarges the cracks and causes incipient disintegration of the surface concrete. This cracking and disintegration progresses deeper and deeper into the concrete, permitting corrosion of the steel reinforcement, and reducing the effective sectional area of both steel and concrete.

Underground concrete surfaces are often exposed to acid and alkaline ground waters which are highly corrosive in their action. These waters are absorbed by the surface pores and are thus brought into close contact with a tremendous area of concrete surface; and, due to the large area affected, dissolution of chemical

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constituents of the cementing material takes place at a proportionate rate, resulting in a porous or cellular concrete structure. The reinforcing steel is also attacked and corroded; and, in the course of time, the underground, concrete structure becomes unfit for service, and expensive repairs or replacement becomes necessary.

It is evident that if concrete structures are to prove permanent and safe, they must be water-tight; and *the condition of many concrete structures today is conclusive evidence that plain concrete alone is not in itself sufficiently water-tight.* Thus the importance of an effective means for waterproofing concrete becomes obvious.

The writer's statements so far as fully confirmed by F. R. McMillan and Inge Lyse of the Portland Cement Association in the Introduction to their paper "Some Permeability Studies of Concrete:"[1] The paper referred to possesses unusual merit; and the co-authors deserve much commendation and respect for the accuracy and frankness which marks their work. The most significant conclusion reached by these two investigators is expressed by them as follows: "The tests have demonstrated what has been observed in concrete structures that defects in placing are a much more important factor in the leakage than ordinary differences in mixtures, materials, or water content." This statement is really an unqualified endorsement of a proper admixture; for it cannot be disputed that such an admixture improves the workability of concrete, and that improved workability operates to prevent defects in placing.

Granting that concrete must be quite workable when placed, if it is to prove sufficiently waterproof for permanence and safety, is then a proper admixture to be preferred to added cement as a means to obtain the required degree of workability? There are several good and sufficient reasons why such an admixture—hydrated lime for instance, is to be preferred:

(1) Lime putty is much more viscous, sticky and slippery than cement paste; hence not only is a lesser volume of the former required for a definite degree of workability, but also volume for volume lime costs only one-half as much as cement. The viscous, sticky lime putty is also more efficient in preventing the separation

[1] JOURNAL, A. C. I., December, 1929 (*A. C. I. Proc.*, Vol. 26, p. 101).

(unmixing) and segregation of the component materials of the concrete; thus guarding against the defects in placing which is the most important factor influencing leakage in the finished concrete. The slippery lime putty acts also as a lubricant coating the particles of the cement and aggregates, and makes them smooth and mobile and easy to chute and otherwise handle into the forms and around and between the reinforcing steel; thus the cost of handling and placing the concrete is reduced and, furthermore, exposed concrete surfaces require no patching or finishing.

(2) The extremely minute lime particles, as compared with cement particles, is another reason for increased impermeability where lime is used; for these minute particles of lime act to fill the void spaces between the cement particles as well as the sand particles, leaving only the most infinitesimal void spaces between the lime particles themselves for the possible passage of water. In addition, these extremely minute lime particles when saturated with water become colloidal or gelatinous (glue-like) in nature, and not only make the lime putty viscous and sticky but also act to seal the infinitesimal void spaces left between the lime particles themselves. The presence of these minute lime particles in the void spaces between the sand and cement particles also makes a stiffer and denser matrix which holds the coarse aggregate in suspension during transportation of the concrete from mixer to forms.

(3) Added cement means a richer concrete mix; and the richer the mix the greater the volume changes in the concrete due to variations in temperature and in moisture content. It has already been pointed out in this discussion, that volume change is responsible for cracking and disintegration of surface concrete; hence lime is to be preferred to added cement, much less lime being required and, also, a mixture of lime and cement being less sensible to temperature and moisture changes than straight cement.

(4) An admixture of lime increases both the strength and the toughness of all but the richest concrete mixes, while added cement increases the strength only.

Hydrated lime is probably the most efficient integral water-tightening medium available today for concrete. Test data accumulated over a period of more than 30 years support this con-

clusion. Portland cement was invented in 1825, but, prior to 1900, very little was used in the United States; hence it can be said that lime has been successfully used as an admixture in concrete from the time portland cement concrete was first recognized in this country. Results of authoritative and unbiased tests made during the 30 year period referred to above, and representing competent evidence of the value of lime as an admixture in concrete, follow:

It seems obvious that the exclusion of reputable and time-tested admixtures from any concrete specification will not serve best the interest of the American public and, that being the case, Paragraph 10 of the "Construction Specification for Concrete Work on Ordinary Buildings" should be amended at once to not only permit but even recommend the use of a reputable and time-tested admixture.

DATA

Tests by Sanford E. Thompson

The textbook "Concrete, Plain and Reinforced," by Taylor and Thompson, is well known to concrete engineers. Sanford E. Thompson, co-author of this book, was one of the earliest advocates of lime in concrete, and used it extensively in his own work. As early as 1903, he made a few permeability tests with hydrated lime as an admixture, and found it a valuable material for water-proofing. In 1906, he made another test which he reports as follows:

TEST FOR: Permeability.
CONCRETE MIX: 1:2:4.
CONCRETE THICKNESS: 4 inches.
WATER PRESSURE: 80 pounds per square inch.

*Hydrated Lime Per Cent	Flow of Water, Grams per Minute		
	14 Days	21 Days	28 Days
0	5.52	2.92	1.91
2	9.20	2.55	1.63
4	2.82	1.49	0.76

*In terms of weight of cement.

From these test data, Mr. Thompson decided that 5 per cent hydrated lime was the proper amount to use; such figures being outside the range of the test, but the minimum possible figure indicated thereby.

Later tests by Mr. Thompson lead him to the following conclusions, presented in a paper before the A. S. T. M. in June, 1908:

(1) Hydrated lime increases the water-tightness of concrete.
(2) Effective proportions of hydrated lime for water-tight concrete are as follows:
For one part Portland cement, two parts sand, four parts stone, add 8 per cent hydrated lime.
For one part Portland cement, 2½ parts sand, 4½ parts stone, add 12 per cent hydrated lime.
For one part portland cement, 3 parts sand, 5 parts stone, add 16 per cent hydrated lime.
These percentages are based on the weight of the dry hydrated lime to the weight of the dry portland cement.
(3) The cost of large waterproof concrete structures frequently may be reduced by employing leaner proportions of concrete with hydrated lime admixture, and small structures such as tanks, may be made water-tight.

These conclusions are based in part at least on the following:

TEST FOR: Permeability.
CONCRETE MIX: See Table.
WATER PRESSURE: 60 pounds per square inch.

*Lime Per Cent	Concrete Age Days	Test Period Hours	Flow of Water Grams per Hour
1:2:4 Concrete:			
1	40	4¼	74.8
4	41	5	28.4
7	42	6¼	5.2
10	46	18	1.6
1:2½:4½ Concrete:			
0	45	6	32.5
10	49	11	0
14	50	27	0
1:3:5 Concrete:			
0	50	14	70.6
8	51	17	3.6
14	50	13	0.7
20	53	15	0.7

*In terms of weignt of cement.

Tests by Prof. M. O. Withey

Permeability and strength test, with lime as an admixture, were conducted at the University of Wisconsin in 1920 under the direction of Prof. M. O. Withey. Two different concrete mixes were tested and the following conclusions were drawn from the results of the tests:

(1) Hydrated lime, when added to the cement increased the compressive strength of the concrete which had a slump of 4 inches, about 4 per cent for each per cent of lime added. For the concrete which had a slump of 9 inches, the strength was increased about 6½ per cent for each per cent of lime added.

(2) The 1:2.8:6.5 mix by volume was very pervious. The addition of 4 per cent of hydrated lime increased the imperviousness of the mix very materially. The addition of 8 per cent of hydrated lime produced practically the same effect as the addition of 4 per cent.

The test data for the richer mix show conclusively that the addition of a proper amount of hydrated lime improves both the strength and the water-tightness of even the richer concrete mixes; hence such test data are included herewith.

TEST FOR: Permeability and Strength.
CONCRETE MIX: 1:2.1:4.9.
WATER PRESSURES: 40 pounds per square inch.

*Lime Per Cent	†Water Per Cent	Unit Strength	Flow of Water, Gallons per Hour 0-50 Hrs.	20-50 Hrs.	40-50 Hrs.
0	6.66	2,030	.00023	.00015	.00011
2	6.83	2,235	.00017	.00009	.00006
4	6.80	2,678	.00019	.00012	.00011
6	6.73	2,855	.00017	.00011	.00010
8	6.86	2,470	.00015	.00007	.00007
10	6.87	3,035	.00017	.00012	.00012
0	7.40	1,266	.00030	.00021	.00018
2	7.37	1,884	.00036	.00023	.00023
4	7.30	1,585	.00026	.00016	.00017
6	7.40	1,664	.00022	.00013	.00012
8	7.40	1,893	.00022	.00012	.00010
10	7.43	2,225	.00025	.00016	.00013

*In terms of weight of cement.
†In terms of total weight of dry materials.

Further tests under the direction of Professor M. O. Withey at the University of Wisconsin were conducted in 1921; and the following conclusions were drawn from the results of this check series:

(1) The rate of flow of water into specimens was, in general, decreased by the addition of hydrated lime to the mix. The decrease was also generally greater for 10 per cent of lime than for 5 per cent.

(2) For the gravel concrete series of 4-inch slump the results do not indicate a great advantage in the use of hydrated lime. The set with 5 per cent of lime was somewhat more impervious than the plain concrete, and the set with 10 per cent of lime showed about the same flow as the concrete with no lime.

(3) The series of gravel concretes with a 9-inch slump shows that the addition of 5 per cent of lime slightly decreased the flow of water, and that 10 per cent of lime decreased it to less than half of the flow for plain concrete. As before, the results are variable for the plain concrete slightly more uniform for 5 per cent of lime, and quite uniform for 10 per cent lime. . . . Again the uniform results and decreased rate of flow indicate that hydrated lime increased the water-tightness of the concrete.

(4) The limestone series of specimens gives average results which indicate that the addition of 5 per cent of lime decreased the rate of flow to about one-tenth of the rate for plain concrete, and 10 per cent of lime causes some additional reduction:

(5) These results again show that hydrated lime decreased the permeability of concrete, but do not give any indication of the relative effect of the addition of 5 per cent and 10 per cent of lime. The great gain in water-tightness when 5 per cent of lime is added can be attributed mainly to gain in workability, and partly to the effect of the lime in filling the voids in the mix. . . . The first 5 per cent of lime made the mix workable, and dense cores were obtained. The addition of the next 5 per cent of lime could not be expected, in the case of this aggregate, to cause a corresponding gain in water-tightness.

The test data on the limestone concrete for the intervals 0-100 hours, 20-100 hours and 40-100 hours follow. The rapid leakage in the case of the plain concrete is due to separation and segregation of the component materials.

TEST FOR: Permeability and Strength.
CONCRETE MIX: 1:2.45:6.2.
WATER PRESSURE: 40 pounds per square inch.

*Lime Per Cent	†Water Per Cent	Unit Strength	Rate of Flow, Gallons per Hour 0-100 Hrs.	20-100 Hrs.	40-100 Hrs.
0	6.47	1,580	All specimens leaked rapidly		
5	6.58	1,828	.00201	.00176	.00179
10	6.65	1,912	.00153	.00134	.00234

*In terms of weight of cement.
†In terms of total weight of dry materials.

Tests by Prof. H. C. Berry.

Strength tests with hydrated lime as an admixture were conducted at the University of Pennsylvania in 1921 under the direction of Prof. H. C. Berry. Two different concrete mixes were included in the tests, and each mix tested at a wet and at a dry consistency corresponding to a flow of 190 and 165 respectively on the U. S. Bureau of Standards Flow Table. The data follow:

TEST FOR: Strength.
CONCRETE MIX: 1:2:4

*Lime Per Cent	†Per Cent Water Dry Mix	Wet Mix	Compressive Strength Dry Mix	Wet Mix
¾-inch Crushed Stone Aggregate:				
0	8.7	9.4	2,842	2,049
5	8.9	10.0	2,889	2,842
10	9.5	10.0	3,021	2,540
¾-inch Gravel Aggregate:				
0	7.8	8.9	2,673	2,016
5	8.5	8.9	2,338	2,338
10	8.9	9.3	2,340	2,340

*In terms of weight of cement.
†In terms of total weight of dry material exclusive of lime.

TEST FOR: Strength.
CONCRETE MIX: 1:2½:5.

*Lime Per Cent	†Per Cent Water Dry Mix	Wet Mix	Compressive Strength Dry Mix	Wet Mix
¾-inch Crushed Stone Aggregate:				
0	7.6	8.1	1,842	1,505
5	8.25	8.6	2,116	2,248
10	8.6	9.1	2,022	2,384
¾-inch Gravel Aggregate:				
0	7.5	8.3	1,957	1.631
5	8.3	8.35	1,865	1,716
10	8.2	8.3	1,822	1,914

*In terms of weight of cement.
†In terms of total weight of dry material exclusive of lime.

It should be noted that the "Dry Mix" concrete is much too dry for practical use in the field, and that the addition of lime increased the strength in every case for the "Wet Mix" i. e., a practical field mix. Thus, these results are in entire accord with the findings of Prof. M. O. Withey and Sanford E. Thompson. Many other tests can be cited, the results of which are in substantial agreement with those quoted; however, this discussion of lime as an admixture in concrete will close with the following report on Wacker Drive, a two-level street in the new Chicago Boulevard system.

Concrete in Wacker Drive is designed to withstand the severe exposure to the elements imposed by its location. Hence the specifications required an admixture of 8 pounds of hydrated lime per sack of cement in all exposed concrete. T. A. Evans, Engineer of Design, Bureau of Design, Chicago Board of Local Improvements, is the authority for the following tabulated figures and pertinent comments under date of September 25, 1925.

EFFECT OF HYDRATED LIME ON †WORKABILITY OF CONCRETE MIXTURES

*Lime Per Cent	Concrete Mix 1:1½:3	1:2:4	1:2½:5	1:3:6
0	44	108	169	242
5	33	103	167	202
10	31	81	125	185
15			129	171

*In terms of weight of cement.
†The lower the workability figure the more workable the concrete.

We have poured a portion of the viaduct structure without the use of hydrated lime and have found that the work of handling and tamping around the heavily reinforced portion of the concrete structure was greatly facilitated on the sections on which we have added lime to the mixture, so we are now using lime on the balance of the work.

A more comprehensive report by R. W. Priest, Field Testing Engineer, follows:

Hydrated Lime in Wacker Drive Concrete—(Synopsis)

A certain degree of skepticism is often expressed by engineers and contractors as to the real advantage to be gained by the use of an admixture in concrete. Questions as to the relative strength-deterioration, absorption, permeability, density and workability of concrete with and without an admixture often arise. As Field Testing Engineer for the City of Chicago on the Wacker Drive, the writer has had exceptional opportunity to study this question. The facts obtained by test on this job are believed to be conclusive.

Hydrated lime was used as an admixture in approximately 60 per cent of the 120,000 cubic yards of concrete in this project, the exception being that lime was not used in the concrete placed below ground, such as caissons, sub-girders and pavement foundations.

The concrete on this work was a nominal 1:2.33:3.67 mixture (1:6 specifications). The fineness modulus of the coarse aggregate and the sand were respectively 6.84 and 2.86. The total water allowance was 7 gallons per sack of cement and each batch of concrete was mixed 1½ minutes in a rotary batch mixer. From all concrete poured, 6-inch by 12-inch field cylinders were made at regular intervals and cured under three conditions so that the results express actual conditions in the concrete as placed.

These field specimens were tested for compressive strength at various ages up to one year and additional specimens were tested for absorption and permeability. The results of these tests are now available and it is possible to make a conclusive comparison between the concrete without admixture and concrete containing 8 pounds of hydrated lime per sack of cement.

A careful study has been made of the compressive strength of about 1,000 representative cylinders taken over a period of two years. These thousand cylinders are only about one-sixth of those taken on the job, but they represent identical mixtures and identical conditions, of curing and testing, which are necessary for a fair and conclusive comparison.

Number of Cylinders	Age at Test	Compressive Strength Average per Square Inch
Concrete without lime (1:6 Mix):		
250	7 days	1,632 lbs.
250	28 days	3,176 lbs.
10	1 year	4,362 lbs.
Concrete with 8 per cent lime (1:6 Mix):		
250	7 days	1,720 lbs.
250	28 days	3,214 lbs.
10	1 year	4,630 lbs.

Considering the large number of the tests made under identical conditions, it is conclusive that the addition of lime does not lessen the strength of concrete and, without reducing the water content as is possible with lime, the strength was slightly increased at all ages tested.

In a test to determine the effect of the addition of lime on the absorption of concrete, it was found that these cylinders containing the admixture absorbed an average of 3.23 per cent of water by weight during an immersion of 96 hours. The specimens without the lime gave an average absorption of 4.126 per cent with the same time of immersion.

The investigation also included penetration tests which were on disc specimens 8 inches in diameter and 2 inches thick, 28 days old at the time of test. Ten square inches of the face of these discs were subjected to a water pressure of 22 pounds per square inch for a period of 72 hours. The discs containing no admixture permitted the passage of 15 per cent more water than those containing hydrated lime.

In order to settle, to our own satisfaction, the question of the yield of concrete when an admixture is used, a series of tests were conducted which demonstrated that the use of 8 per cent of lime did not give an increase, but rather resulted in a slightly less volume of concrete than when it was omitted. The writer believes that this work proves that concrete containing lime is more dense than without it.

In conclusion it was observed on the work at Wacker Drive that concrete containing lime was more plastic and workable, and was more easily handled with less chance of segregation of aggregates. To a contractor these features are money savers.

From the Engineers' standpoint it was observed that with lime, less water was needed to make a workable mixture. The tests showed increased strength, decreased absorption and decreased penetration; all of which follow logically the demonstrated greater density of lime concrete.—R. W. PRIEST, Field Testing Engineer, Wacker Drive; Board of Local Improvments; Bureau of Design, Chicago.

AUTHOR'S CLOSURE*

MR. BROWN'S discussion† was not received by me prior to the convention and was not discussed there. I am aware of the apparent hardship that disheartens him. But I still think that the Contractor, as the most experienced person connected with the detailed operations of construction work, should continue to provide "all usual necessary and essential portions" of the work even though not shown or specified completely. If he is in doubt in making up his estimate—the only complete list of every item required—as to whether lintels are stone, terra cotta or concrete, he should find out. On the ordinary building that is not a difficult thing to do and the Contractor is in far better

*By ARTHUR R. LORD, Lord & Holinger, Chicago.
†H. M. Brown, JOURNAL, March 1930. (*A. C. I. Proc.* Vol. 26, p. 580),

position than any one else to do this necessary safeguarding work.

The second paragraph in Article 11 deals with "gaps" in the reinforcement—small unreinforced areas such as may occur on any job either by oversight of the detailer or by careless steel placing. It has nothing to do with the spacing of bars in the regular bands or systems, but is concerned only with "gaps" where the regular systematic groupings of bars leave an unreinforced opening. If this opening were large, or if there were many of them, I assume that the Contractor would call in the Engineer to complete his design and that he would be paid for any considerable amount of additional steel required. But for the small unreinforced triangle or space indicated by the word "gap," no such elaborate or time consuming action is desirable—simply put in $\frac{3}{8}$ in. at 6 in. o. c. and pass on. This is the simplest and easiest way to cover such minor openings and the amount of steel consumed is negligible in my experience. This provision was added by a highly experienced critic.

Placing concrete under water is quite common in buildings involving cylindrical piers or "caissons" of concrete, which is a common type of foundation in our larger cities today. I have never seen bottom dump buckets used in such work but this device was nevertheless inserted in the specification before its adoption as a tentative standard by the Institute. So anyone may use this method who wishes to when operating under this specification.

One function of a specification of this sort is to protect owners from "extras" in cases where a reasonable contact of the bidder with the Architect or Engineer would eliminate uncertainties. I assume, and the lack of any considerable amount of unfavorable criticism of this specification appears to support the assumption, that Contractors want to stay on good terms with their clients and wish to cover in their bids all required work in their line, rather than to catch the Owner unawares with extra charges for a lot of petty items. These specifications follow the time-honored method of accomplishing this end—it makes the contractor financially interested in eliminating uncertainties before entering into contract.

Whenever it has seemed possible and proper I have, with the consent of the critic members of the committee, made the appropriate changes in the wording of this specification, as suggested by the discussion received. Many changes were submitted to the convention in 1930 and incorporated in the specification as tentatively adopted by the Institute. The recent discussion, received since the convention, has centered around article 10. With no thought of malice, I submit that the authors of this discussion have not read this article. Doubtless they meant to read it, but their discussion reveals that they did not read it soberly and with the wish to understand. It does not seem to me to be obscure. Article 10 does not prohibit the use of admixtures. On the contrary, it specifically provides for the use of such admixtures as come with a satisfactory record of use, or even those that can win the approval of the architect or engineer—a none too difficult feat surely. It does, by its negative form of statement, act to restrain the contractor from rushing into the arms of the first plausible admixture salesperson that he chances to meet.

There are today upward of 250 materials, or rather packaged products, the producers of which would like to have incorporated in concrete. It is a natural wish. So much concrete is used that even a small amount in every yard of concrete would make a nice business total. Some of these "admixtures" have been extensively used, others hardly used at all. Some of them are doubtless desirable additions to concrete for certain uses, common uses, as workability agents or waterproofers. Some of them are pure gambles. Article 10 requires evidence that admixtures used on work done under Specification 502 have a history as well as a sales agent. It is designed to protect the manufacturer of a worth-while admixture from unworthy competition. Evidently lime, one of the longest-used of this general class of material, does not wish protection. But the owner is entitled to protection in his specification, even if the manufacturer scorns it. I believe that Article 10 should stand, both for admixtures (including waterproofing compounds) and for the new special cements.

Mr. Ellsworth's leading witness in favor of lime is a job on which I personally approved its use, for the conditions holding on

that job. I rejected in the neighborhood of fifty other admixtures that were pushed for use on the same job, most of them depending chiefly on political backing rather than on the kind of evidence required by Article 10. If Mr. Ellsworth and Mr. Hutchinson will cool off long enough to read Article 10 quietly, I believe that they will agree that it is a common sense provision in a field where there are so many products of so diverse merits. It seems to me also that they might agree that even lime isn't the best admixture to use for all concrete, and that some concrete does not require any help from admixtures. Lime isn't altogether indicated for the concrete of a dam intended to impound lime-thirsty mountain water. Article 10 puts the decision on the architect or engineer where it properly rests. If this article results in worth-while materials being advanced on their scientific merits and proven results, instead of on the fallacious theory that concrete is a fair dumping ground for any material that does not stop the mixer, it will be worth a few misunderstandings such as these, much as I regret them.

Discussion of the Report of Committee 802

"Good Practice in Concrete Floor Finish"*

BY ROBERT SETH LINDSTROM†

The writer wishes to submit further discussion, so that when and if a specification is formally prepared and submitted to the Institute for adoption, all of the kinks will have been ironed out and a floor finish specification presented that will be accepted by the architect-engineer and building construction industry for universal use.

Portland cement is the tie that binds the ingredients entering into portland cement concrete, solidifying the structural mass to a point where concrete has been generally accepted in reinforced concrete construction. In wearing surfaces of portland cement concrete subjected to traffic as in floors, ramps, and stairs, there must also be a compact, homogeneous finish.

To obtain the densest possible concrete for floor finish the mix, sizes, shape, and quality of the aggregates must be considered more carefully than aggregates for structural concrete due to the experience that if the aggregate is not properly and uniformly graded from coarse to fine, voids will occur and the density of the floor topping be impaired.

Nothing has done more harm in lessening the use of concrete floors than the method called "Dusted-on Finish."

In a dusted-on finish all the rules and regulations of good practice are violated and the water-cement ratio lost sight of.

We have all learned by our field experiences that the proper amounts of water, cement and graded aggregate must have such a period in a mixer as will thoroughly mix water, cement and sand so that when leaving the mixer the mass will not segregate in handling or placing. Additional cement, sand or other ingredients thrown or dusted on in a dry state cannot cover the

*Journal, A. C. I., March, 1930; *A. C. I. Proc.*, Vol. 26, p. 520.
†General Manager, Advance Waterproof Cement Co., Chicago, Ill.

surface uniformly nor will there be sufficient water properly to take up the cement for a uniform, plastic condition like the material that has passed through the mixer.

Since standard portland cement must pass a certain percentage through a 200 mesh sieve it is necessary to use a fine aggregate grading, beginning nearer the fineness of the cement and gradually increasing the size to an ideal sand grading, meeting with Fuller's formula for ideal sand grading. This formula produces a grading as follows:

4	Mesh to the inch passing	100%
8	" " " " "	75%
14	" " " " "	50%
28	" " " " "	25%
48	" " " " "	15%
100	" " " " "	0

Such fine-to-coarse aggregate with portland cement and water, machine mixed for fully two minutes, properly laid and troweled will give a 100% non-dusting portland cement floor topping and any excess of cement and sand dusted on will change the entire mix and create a dusty, non-uniform mixture that will add to the cost of the work.

Field practice and years of tests have shown conclusively that the old 1:2 mix, taking into consideration amount of water and portland cement, together with size and grading of aggregate, should be standardized and offered to the committee that will formally and finally draft floor finish specifications.

Construction of Main Canal Lining on Kittitas Division, Yakima Reclamation Project, Washington

BY ARTHUR RUETTGERS AND A. A. WHITMORE*

Construction work performed on the Kittitas Division of the Yakima Reclamation Project during the seasons of 1927 and 1928 included the placing, by contract, of about 55,000 cu. yds. of concrete in canal and tunnel lining, combination lining and flume sections, inverted siphons, culverts, turnouts, bridges, and miscellaneous canal structures. The greater portion of the concrete, about 52 per cent, was placed in 3-in. reinforced canal lining. A description of this work, which involved the successful development and use of some novel construction equipment and the application of the more modern methods for scientifically controlling the concreting operations, forms the main substance of this paper.

LOCATION, PURPOSE, AND GENERAL DESCRIPTION OF WORK

The Kittitas division of the Yakima project will, upon completion, provide facilities for the irrigation of about 72,000 acres of land in the higher reaches of the bowl-shaped Kittitas Valley, surrounding privately-irrigated bottom land centered at Ellensburg, Washington. The irrigation plan contemplates the diversion of 1,320 second feet of water from the Yakima River at the Easton dam, about 40 miles northwest of Ellensburg. The Main Canal, now completed, traverses the southern slope of the Yakima River Canyon for 26 miles to its entrance into the Kittitas Valley, where the canal divides—the north or larger branch (of 925 second feet capacity) crossing the river and skirting the

*Arthur Ruettgers, Engineer, Denver; A. A. Whitmore, Associate Engineer, Ellensburg, Wash., U. S. Bureau of Reclamation.

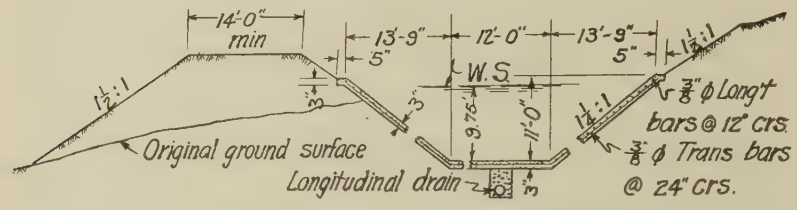


FIG. 1—TYPICAL SECTION CANAL LINING—NOT TO SCALE



FIG. 2—LOOKING UPSTREAM FROM STATION 1202 MAIN CANAL; TWO DRAGLINES EXCAVATING FOR LINED SECTION

northern and eastern rims of the valley, and the south branch (of 250 second feet capacity) following along the southern rim.

Stretches of the Main Canal, aggregating about half its entire length, were lined with 3-in. reinforced concrete to prevent excessive seepage losses in sand and gravel formations, to reduce the construction cost through heavy cuts, or to serve as a safeguard against expensive washouts on steep side-hill locations. Construction of the lined canal was performed under five separate



FIG. 3—TRIMMING AND LINING OPERATIONS; DRAGLINE EQUIPPED WITH CLAM SHELL BUCKET USED TO REMOVE EXCESS MATERIAL FROM TRIMMING OPERATIONS

FIG. 4—TEMPLATE USED IN TRIMMING OPERATIONS. LOOKING UPSTREAM FROM STATION 105 MAIN CANAL. OUTLET OF TUCKER CREEK SIPHON IN THE BACKGROUND



FIG. 5—APPARATUS USED BY DERBON CONSTRUCTION CO. FOR TRIMMING OPERATIONS ON DIVISION 3, MAIN CANAL. A SCREED BOARD PROVIDED WITH A STEEL CUTTING EDGE IS DRAWN DOWN THE SLOPE, THE ENDS OF THE CUTTING SCREED REST ON BOARDS SET TO GRADE OF CANAL SIDE SLOPE

contracts by four different contractors. A typical section of the canal is shown in Fig. 1.

WORK PRELIMINARY TO CONCRETING

Excavation. All rough excavation of the canal section was performed by draglines equipped with buckets of 1 to 2½ cu. yds. capacity. (Fig. 2). Smaller draglines, usually equipped with clam-shell buckets, were used to dispose of excess material removed in the trimming operations. (Fig. 3).

Trimming. The final trimming was generally done with the aid of a traveling template (Figs. 4 and 5), mounted on rollers, and consisting of two wooden trapezoidal-shaped frames so constructed and placed with respect to the canal center line and grade that a straight-edge moved along between the frames indicated the proper subgrade for the side slopes and canal bottom. The roller track was made of 3-in. by 6-in. plank

firmly embedded in the canal bottom, adjacent to the toe of each side slope. (Fig. 9). Removal of the high spots in the canal section and backfilling of the cavities were performed by hand methods. Cavities deeper than 6 in. were backfilled by placing moist material in thin horizontal layers and compacting with standard iron tampers, otherwise the compacting was done with flat steel shovels. Where the material was excessively sandy a thin layer was removed and replaced with clay to prevent ravelling of the surface. Further efforts to prevent ravelling of finished surface consisted in keeping the subgrade moist by sprinkling until the concrete was finally deposited. (Fig. 6).

The traveling template was also utilized in providing narrow berms at the tops of the side slopes. A ribbon of plank, placed on these berms, served as a second track for the operation of a movable scaffold from which the reinforcing steel on the side slopes was tied (Fig. 6) and subsequently to guide and support the concrete distributing platform and lining machines.

Drainage. Inasmuch as a 3-in. concrete lining in a canal section of the dimensions used offers little resistance to external pressure when the canal is empty, and as the region traversed is subject to a mean annual precipitation of some 20 ins., with the major portion occurring during the non-irrigation season, particular care was taken to provide adequate drainage for the lining. Longitudinal drains of 4-in. to 12-in. vitrified or concrete pipe, embedded in gravel, (Fig. 7) were laid beneath the floor of the lining in one or two lines, depending on the topography and ground water conditions encountered in excavating. Where the conditions appeared to require it, feeder or "finger" drains of gravel or tile, at right angles to and connecting with the longitudinal drains, were placed in the upper side slope of the lining. The importance of these finger drains, particularly at draws, was emphasized during the progress of the work by the failure of several panels on the upper slope through the action of hydrostatic pressure back of the lining. (Fig. 8). The early practice was to place the longitudinal drains along the center line of the canal but this was abandoned in favor of locating the drains near the toe of the upper slope, the latter location offering more



FIG. 6—LINING OPERATIONS MAIN CANAL STATION 1094 ±. MOVABLE SCAFFOLD FOR TYING REINFORCING STEEL IN FOREGROUND. LINING MACHINE, CONCRETE DISTRIBUTING PLATFORM AND MIXER IN BACKGROUND, THE SUB-GRADE BEING WET DOWN PRIOR TO PLACING CONCRETE LINING



FIG. 7—CONSTRUCTING LONGITUDINAL DRAIN. TRIMMING "JUMBO" IN BACKGROUND. DRAGLINE EQUIPPED WITH CLAMSHELL BUCKET USED TO REMOVE EXCESS MATERIAL LEFT FROM TRIMMING OPERATIONS

direct connection with finger drains and better prospects for effective drainage in general. (Fig. 9). The longitudinal drains sloped in both directions to lateral or outlet drains, spaced 300 to 1,000 ft. or more apart and consisting of 6-in. to 12-in. pipe, with cemented joints, placed through or under the lower canal bank. (Fig. 10).

CONCRETING OPERATIONS

Requirements. The matter of devising the most suitable means for handling the concreting operations was not a simple one, as the character of the work and related specifications

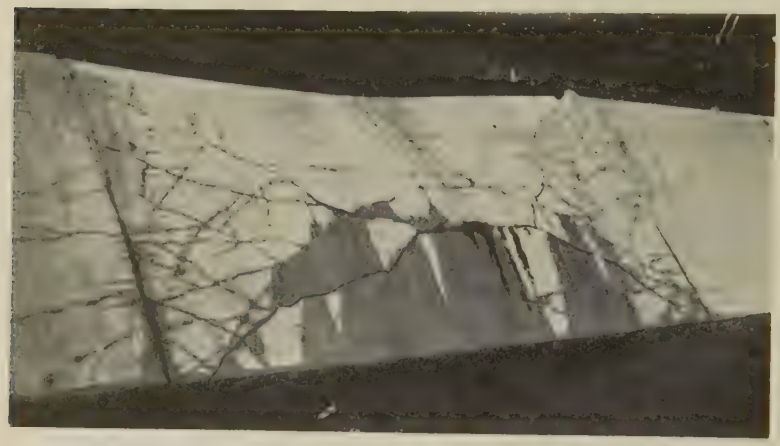




FIG. 8—BREAK IN CANAL LINING CAUSED BY DEVELOPMENT OF HYDROSTATIC HEAD BEHIND THE LINING. STATION 90, MAIN CANAL

FIG. 9—GRAVEL "FINGER" OR SIDE SLOPE FEEDER DRAINS. (NOTE THE LONGITUDINAL DRAIN PLACED NEAR UPPER SIDE OF CANAL BASE.) ALSO NOTE RIBBONS AT THE TOP OF SIDE SLOPES AND ON BOTTOM OF CANAL ON WHICH THE CONCRETE LINING PLATFORM RUNS. LOOKING UPSTREAM FROM STATION 1224—MAIN CANAL



FIG. 10—DISCHARGE FROM LATERAL DRAINS—TWO 6-INCH PIPES DISCHARGING ABOUT ¾ SEC. FEET—STATION 140—MAIN CANAL

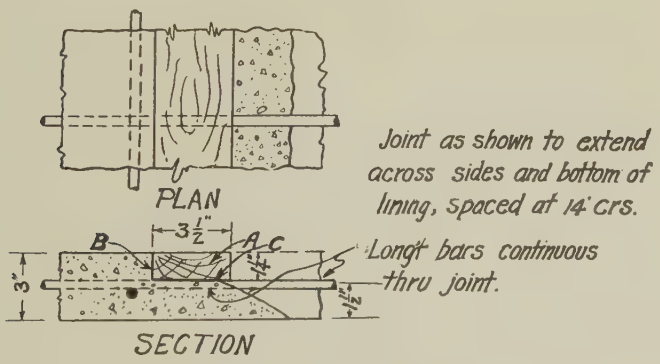
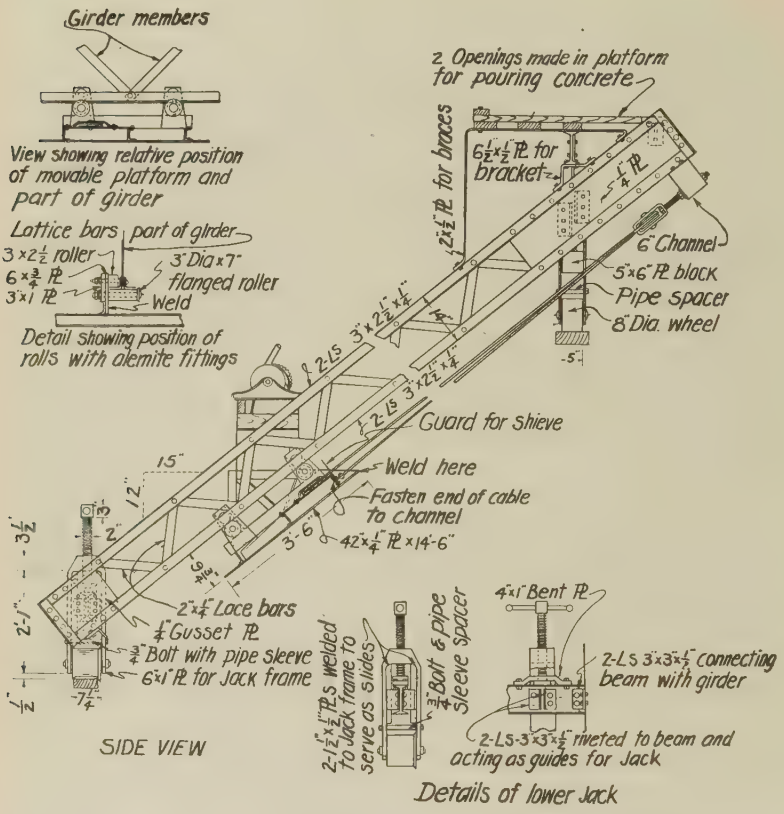


FIG. 11—DETAIL OF JOINT



required that account be taken of a variety of conditions, some of the principal ones being as follows:

(a) The size of the lined section, steepness of side slopes, small thickness of concrete, and difficulty of centering the reinforcing steel in the thin concrete slabs were not favorable for the adoption of the customary method of hand placement of the concrete.

(b) The top of the lower bank (of 14 ft. minimum width), which as a rule furnished the only means of direct access to the work, provided only a limited space for the storage of materials and the accommodation of operating equipment and crews.

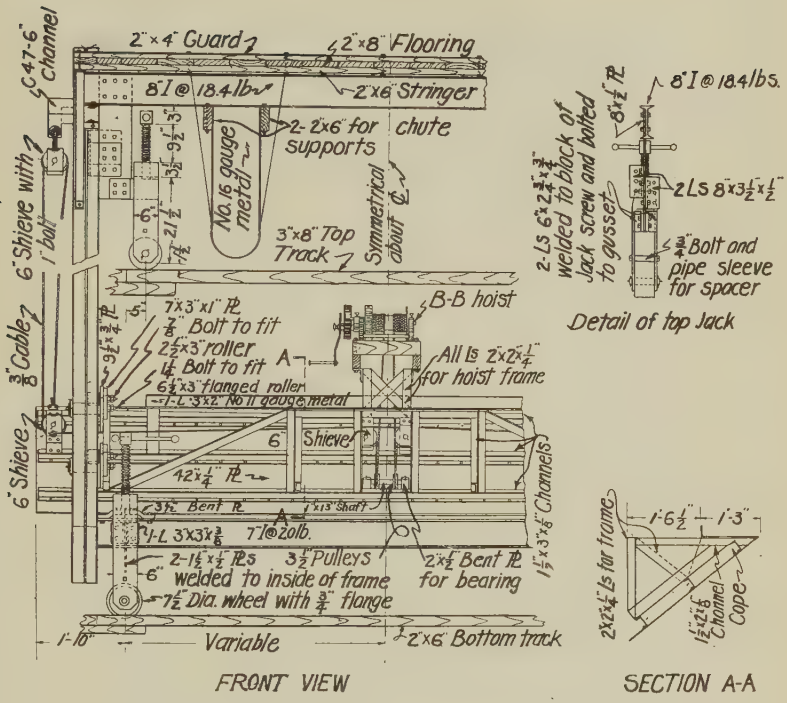


FIG. 12—MOVABLE FORM FOR PLACING CONCRETE LINING

(c) The distance over which the work was spread, and the necessity for completion within the allotted time, demanded mobility of all operating equipment. Simultaneous prosecution of concreting operations in several locations involved duplication of equipment and organization.

(d) The specifications required that the lining be placed monolithically in short panel sections and given a steel trowel finish. A special transverse joint (Fig. 11) later abandoned and replaced by a plain butt joint as hereinafter discussed was required at panel ends. Longitudinal joints (parallel to the axis of the canal) were not permitted.

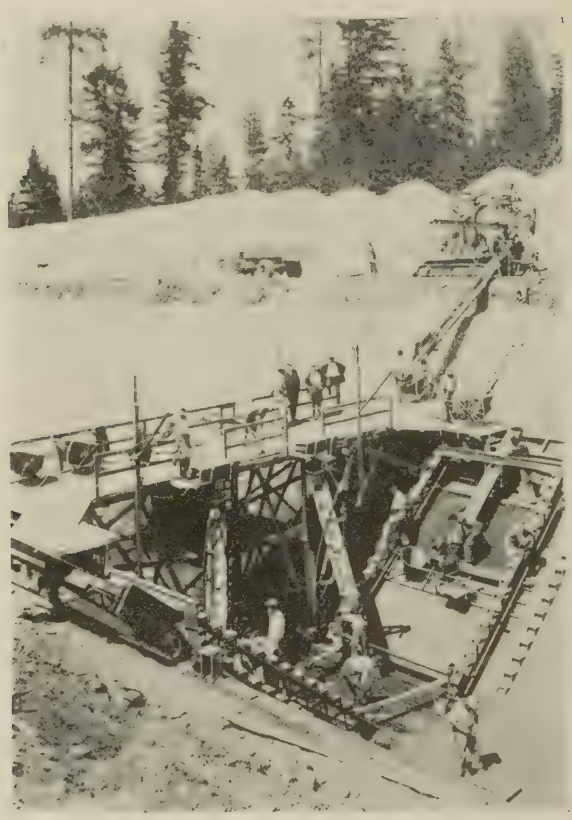


FIG. 13—ORIGINAL LINING MACHINE AND CONCRETE DISTRIBUTING PLATFORM DEVELOPED BY GENERAL CONSTRUCTION CO., CONTRACTOR FOR DIVISIONS 1 AND 2 MAIN CANAL. NOTE TRUCK ABOUT TO BACK UP AND DISCHARGE CEMENT AND AGGREGATES INTO HOPPER OF THE CATERPILLAR TRACTION PAVING MIXER. EACH TRUCK HANDLED 3 BATCHES OF DRY MATERIALS

(e) Neighboring creeks and the Yakima River furnished the only adequate and reliable sources of water.

(f) The bid prices were such that efficiency of operation and exercise of the strictest economy were essential for the realization of a profit.

The extent to which the first contractor succeeded in adapting his concreting operations to the particular requirements of the job, in lining approximately 2½ miles of Main Canal in 1927, is evidenced by the fact that all three of the other contractors who engaged in lining operations in 1928 employed methods and equipment embodying the same general principles, insofar as the handling and placing of concrete is concerned, as those which characterised the 1927 work.

Steel plate lining machine. Of particular interest is the novel machine designed and perfected by the forces of the General Construction Co. for lining the side slopes. (Fig. 12). It consists of a movable steel plate 14 ft. 6 in. long (the customary panel length being 14 ft.) and 3 ft. 6 in. wide, supported by and operating under two parallel fabricated steel girders with roller ends. (Figs. 13 and 14). An operating platform about 18 in. wide attached to the upper edge of the steel plate received the concrete and accommodated several workmen, in addition to carrying the winch by means of which the plate was made to travel upward along the canal slope and compact the concrete as it was spaded into the space back of the upper portion of the plate. A screw jack at each roller for raising or lowering the girders permitted adjustment of the plate to the required slope angle and thickness of concrete. The simultaneous use of two of these machines, one on each side slope, not only fulfilled the requirement for monolithic construction of panel units (the floor section being placed by hand immediately following completion of the slope sections) but also introduced an element of competition between the individual working crews which had a noticeable effect in speeding up the work.

Transporting and placing concrete. Wheel buggies were generally used to transport the concrete from the mixer, on the lower bank, to the canal section, where each batch was directed to its desired position on the side slope or bottom with the aid of a distributing platform mounted on rollers and equipped with a series of flexible chutes or spouts. The ribbons of plank track in the canal bottom and at the top of the lined slopes also served as screeds and, in the latter case, as outside forms for the 5-in. concrete berms.

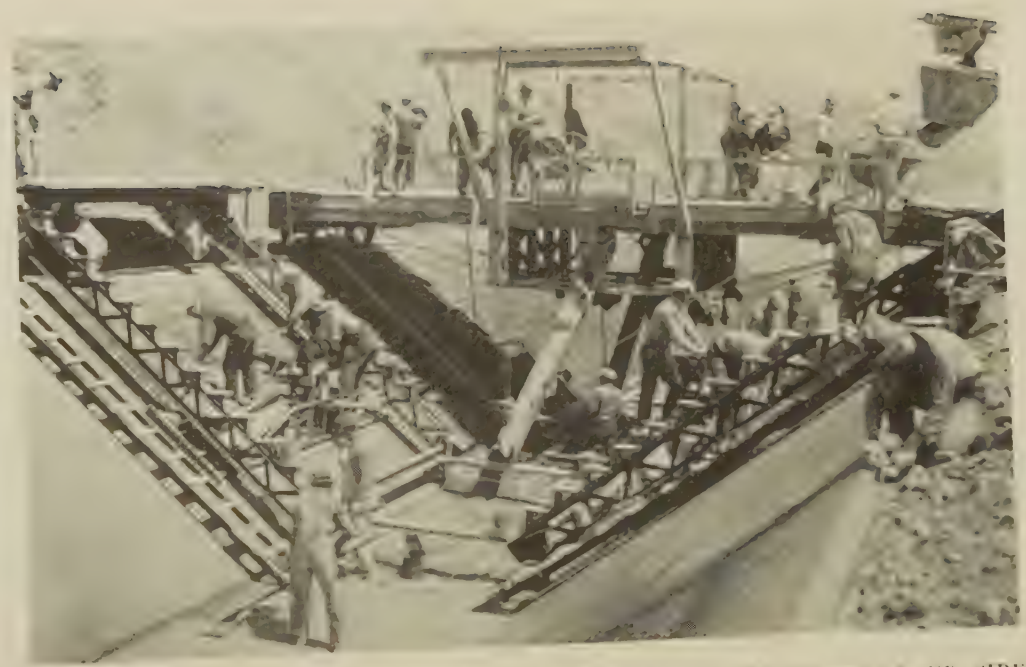


FIG. 14—LINING MACHINE SHOWING MOVABLE STEEL PLATE TRAVELING UP SIDE SLOPE. LINING IN PRECEDING PANEL BEING FINISHED AND CONCRETE BEING PLACED IN LOWER 31_2 FEET OF SIDE SLOPES FOR SUCCEEDING PANEL. NOTCHED SCREED BOARD ON SIDE SLOPE UNDER "JUMBO"

About 10 complete panel units (equivalent to 140 linear feet of lining) constituted the average pour for one working shift of 8 hours, the record for a shift being 14 panels or 196 ft. of lined canal. The reinforcing steel was kept above the subgrade by inserting small pre-cast cement mortar blocks underneath the bars at various points as conditions required. Experience proved that the most practicable procedure for pouring the lower 3½ feet of slope lining was to shovel the concrete directly on to the slope (Fig. 14) and, after striking it off roughly, to lower the steel plate to position and rap it with a bar in order to bring the fine material into contact with the plate. Upon completion of this operation, the plate was started on its upward course, its movement keeping pace with the addition of concrete from the platform by the spaders. The action of the moving steel plate, with proper spading, correct concrete consistency and other related factors under control, resulted in a true surface and a reasonably good finish. The finish was perfected by going over the entire surface lightly with a long-handled steel trowel. Any defects appearing on the surface, due to insufficient mortar or spading, were remedied by adding the amount of concrete or mortar needed and trowelling over. Mortar was also used to provide fillets at the intersections of the floor and side slopes. The floor, as previously indicated, was placed by the usual hand methods, one of the last operations being to remove the plank track or screeds and fill in the grooves with tamped concrete after thoroughly cleaning the grooves (Fig. 14).

Other types of lining machines. Photographs reproduced in Figs. 15 and 16 show types of lining apparatus in which the general principles of the movable plate lining machine and distributing platform were combined in a single unit. These forms of apparatus involved a smaller plant investment and were devised by two of the contractors whose lining operations were less extensive. As seen from the pictures, the movable steel plate in the original slope lining machine was replaced by a metal-lined plank strike-off board, with operating platform, which, instead of being rigidly held down at its ends, simply



FIG. 15—COMBINED DISTRIBUTING PLATFORM AND MOVABLE STRIKE-OFF BOARD TYPE OF APPARATUS EMPLOYED BY C. F. GRAFF, CONTRACTOR FOR CONSTRUCTION OF EASTON DAM AND UPPER HALF-MILE OF KITTITAS MAIN CANAL

rode on two plank screeds spaced a panel-length apart and under the distributing platform, or "jumbo," as it was called.

In comparison with the steel plate lining machine, the spading board was less effective in compacting the concrete and drawing mortar for finishing to the surface, consequently more hand-finishing and a greater use of mortar was required. This feature of the spading board type of apparatus, coupled with its lack of headroom, greater dependence on a definite consistency of concrete for satisfactory operation, and other characteristic features tended to reduce the speed of concrete placement below that obtained by the use of the more expensive type of equipment, although it is believed that the quality of the lining secured was considerably better, and the cost to the contractor less, than would have resulted under the ordinary methods of hand placement.

Concrete materials. The materials entering into the lining construction, including sand and gravel shipped from Puget

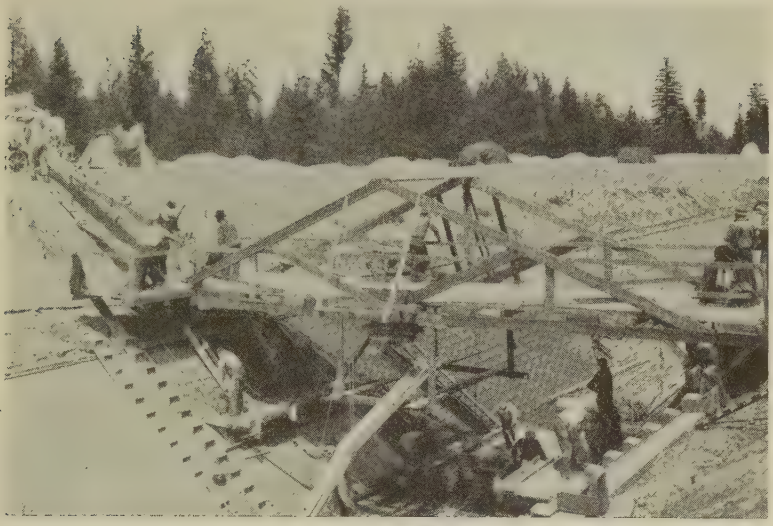


FIG. 16—COMBINED DISTRIBUTING PLATFORM AND MOVABLE STRIKE-OFF BOARD EMPLOYED BY S. H. NEWELL & CO., CONTRACTOR FOR CONSTRUCTION OF DIVISION 2 (SCHEDULE 2), MAIN CANAL. CONTRACTOR STOCK PILED THE CEMENT AND AGGREGATES ON THE CANAL OPERATING ROAD

Sound, cement, diatomaceous earth admixture, reinforcing steel, and drainpipe were received by the contractors at the most convenient railroad points and hauled by them in motor trucks to the work. As a rule, the aggregates were dumped into stock piles, from elevated spur tracks provided by the contractors at the railroad delivery points, and loaded into trucks by mechanical loaders, or elevated to storage bins of central proportioning or mixing plants.

Concrete mixing. Concrete for the first half-mile of lining on the Main Canal was mixed at a stationary mixing plant (constructed primarily for the production of concrete for the Easton diversion dam) and hauled to the scene of lining operations in narrow-gage cars drawn by dinkey locomotives. With this one exception, all concrete placed in Main Canal lining was mixed in portable mixers which traveled the canal bank along with the concrete placing equipment. Modern paving mixers with cater-

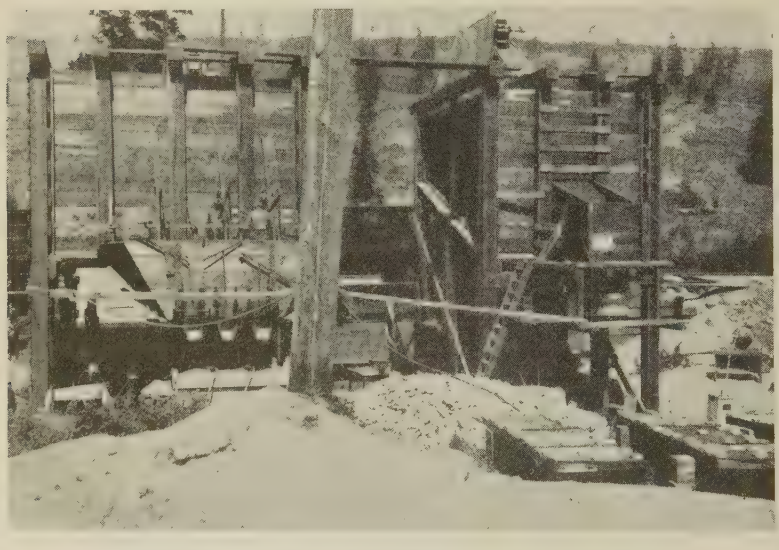


FIG. 17—DERBON CONSTRUCTION COMPANY'S SAND AND GRAVEL BUNKERS AT DERBON'S SPUR ON THE CHICAGO, MILWAUKEE, ST. PAUL & PACIFIC RAILROAD ABOUT 1½ MILES WEST OF HORLICK

pillar traction were employed on some of the more recent work. (Fig. 13).

On two of the contracts the mixer was fed from stock piles of materials on the top of the bank, supplemented, where necessary, by direct motor truck deliveries from the main sources of material supply. On the remaining two contracts, the contractors established central proportioning plants at the railroad delivery points (Fig. 17) where the requisite aggregates, cement and admixture for a mixer batch were measured out and loaded into trucks with bodies specially constructed to handle three batches of the dry materials and, upon arrival at the mixer, to unload the batches separately into the charging hopper.

Water problems. The necessity for almost constant sprinkling of the subgrade from the time of final trimming until the concrete was laid, the use of water for mixing concrete and supplying camp needs, and the specification requirement for keeping the freshly placed concrete continuously damp for a minimum period

of 14 days created a heavy and steady demand for water. This demand was met, on four of the contracts, by piping from neighboring creeks or the Yakima river, usually with the aid of pumps, and, on certain sections of the work, by diverting the creek flow into completed portions of the lined canal and impounding it behind a small earth dike in the canal section. On the fifth and largest contract, an adequate creek flow was not available, and water for all purposes except curing was obtained by pumping from the Yakima river, through approximately 600 linear feet of 3-in. pipe and against a head of about 350 ft. to an elevated storage tank adjacent to the canal, whence it was pumped through smaller pipe of varying lengths to the particular locations required.



FIG. 18—CURING CANAL LINING BY WATER METHOD, USING BABB SPRINKLERS. BURLAP COVERING THE GREEN CONCRETE IS SHOWN IN THE BACKGROUND. STATION 103, MAIN CANAL

Concrete curing. Curing of the concrete by the water method was effectively accomplished by the use of Babb sprinklers (Fig. 18) at short intervals along two lines of pipe paralleling the canal center line—one on each side slope. Operation of the sprinklers was continuous for 14 days after the date of pouring, except for short night periods in certain instances. The lining was completely covered with a layer of burlap during the first seven days of curing, which served principally to protect the green concrete



FIG. 19—COMPLETED CANAL LINING. CURED BY HUNT PROCESS PAINT METHOD. PAINT IS SPRAYED ON IMMEDIATELY FOLLOWING THE FINISHING OPERATIONS

from the direct rays of the sun and distribute the applied moisture more uniformly.

A substitute for the water sprinkling method was employed in curing the lining in the last 6 miles of the Main Canal. The method used (Hunt Process), which was based on the principle of conserving the original mixing water for cement hydration, consisted in coating the exposed surfaces of the fresh concrete with a special asphaltic compound. The material was applied with a spraying machine (Fig. 19), immediately following the finishing operation.

Panel joints. Considerable trouble was experienced, during the earlier lining operations, in obtaining a satisfactory transverse construction joint between adjacent panel units. Details of the type of joint originally contemplated, and employed on other projects for lining of 4-in. or greater thickness, are shown in Fig. 11. The theory of the joint is that when the plank form A is removed from the partially hardened concrete (shown in

section) and fresh concrete is added, a good bond should develop with the irregular shaped tongue C, but the formed vertical surface B should bond very imperfectly and act as a plane of weakness to induce a crack. This result was not realized in practice, the cracks frequently taking a zig-zag course within the limits of the tongue, or a fairly regular path along a plane other than the one intended. The small thickness of the lining, the difficulty of obtaining the vertical surface and holding it after removal of the plank, and the apparent impracticability of securing a substantial tongue of concrete, are some of the reasons attributed for failure to produce a satisfactory joint of the type described. Various attempts were made to improve the joint. One was to leave the plank in place until the second pouring was completed and then fill in the groove, but this resulted in the formation of cracks along both edges. The best results were in using end plank or screeds of the full lining thickness and notched to accommodate the interfering longitudinal reinforcing bars (Fig. 15), thus eliminating the troublesome tongue of concrete and producing a plain butt joint. With the introduction of the butt type of joint, the top surface of the concrete at the joint was penetrated to a depth of about ¾-in., with a special tool similar to a sidewalk jointer, while the concrete was plastic, and the operation repeated after the concrete had stiffened. Cutting the joint was early discontinued, upon discovering that it was not required to insure the development of the crack at the joint. Use of the cutting tool, however, produced an excellent joint in that rounded shoulders were formed each side of the joint which do not spall under slight movement of lining resulting from temperature changes.

Transverse cracks. The lined section and construction joint were designed with the view of controlling the inevitable development of transverse cracks by inducing their occurrence at short regular intervals and along straight sightly lines. That this control was not fully realized with the perfection of the construction joint was evidenced by the appearance of additional intermediate transverse cracks within several days to six weeks after the concrete was placed. A survey of side-slope cracks along selected stretches of completed lining showed that the number of

intermediate cracks averaged 0.8 per 14-ft. panel for the water-cured lining and 1.3 for that cured with the asphalt coating. In other words, taking account of the regular construction joint cracks, the transverse cracks occurred at average intervals of 8 ft. and 6 ft. for the water-cured and asphalt-treated lining respectively—indicating that the amount of longitudinal reinforcing steel used might have been in excess of that required to restrict the contraction cracks to the 14-ft. spacing, under the particular conditions prevailing. Three principal reasons advanced for the increased number of intermediate cracks in the asphalt-treated lining (which increase was more marked on the north or sunny slope) are (1) the apparent greater surface drying through evaporation, (2) the more rapid setting-up of the concrete, permitting less adjustment between the concrete and reinforcing steel during the setting period, and (3) the probable greater daily range of temperature variations in the concrete, due to the black color of the surface coating.

Another significant feature revealed by the survey was that the first panel placed each day exhibited a tendency to develop from 1½ to 3 times as many intermediate cracks as the panels placed subsequently. Evidence obtained from an investigation of the possible causes of this phenomenon indicated quite conclusively that the restrained condition of the longitudinal steel in the first panel, by virtue of its embedment in the hardened concrete of the previous day's pour, was chiefly responsible for the excessive cracking tendency—the theory being that the rigid steel immediately adjacent to the hardened concrete offered greater opposition to the contraction forces in the new concrete, and consequently forced cracks at closer intervals.

CONCRETE MIXES, TESTS, AND CONTROL METHODS

Need for control. The liberal appropriations available and in prospect at the time construction of the canal and distribution systems of the Kittitas division was inaugurated offered favorable prospects for the prosecution of work on a rather large scale. It was early recognized that the production of uniform concrete of the best quality for the money was dependent upon a wise choice of aggregates and the development of a carefully prepared plan of procedure for concrete design, inspection and control.

The vital importance of securing quality concrete, as well as the justification for a reasonable amount of investigational and directional work leading to this end, became manifest when it was realized that estimated expenditures for the production of concrete for canal lining and structures represented approximately 50 per cent of the total proposed construction investment of $9,000,000.

Investigation of aggregates and concrete mix. Attention was first directed to the selection of suitable aggregates. This involved the preliminary testing, by the Bureau of Standards, of numerous samples of materials taken from local deposits, followed by an exhaustive test of further samples from the one local deposit which promised to be satisfactory. The latter test was conducted at a laboratory which had been established at the project office at Ellensburg and equipped to make the customary physical analyses, mortar briquette tests, and the 6 x 12-in. standard cylinder compression tests. Additional apparatus was subsequently installed for making impermeability tests of concrete discs 12 in. in diameter and 4 in. thick under water pressure. Before completion of the tests the aggregates exhibited some questionable characteristics and decision was finally reached to use a high quality sand and gravel from the Puget Sound region.

The specific aggregates for use having been selected, an extensive series of laboratory tests was carried on to determine the gradings and maximum sizes of aggregates best suited to meeting the various concrete requirements, to develop water-cement ratio strength control curves for the range of mixes contemplated, to establish the water-pressure-resisting properties of the different mixes, and to acquire other information of value such as the relation between strength, density of concrete, and density of dry mixed aggregates for various gravel to sand ratios, the effects of varying periods and kinds of curing, the resistance of concrete to weathering, and last but not least, the economic relationships. Standard specifications were followed in all laboratory procedure, except where such specifications were not available, as in the case of the impermeability tests. In practically all cases the laboratory test specimens were made with 2-in. and 6-in. slump concrete, this being the range of consistencies permitted by the construc-

tion specifications. The gravel to sand ratios used in the mixes first designed were based on the recommendations of Duff A. Abrams. These were modified somewhat after thorough trial in the field, for it was found that an increase in the proportion of sand, especially in the richer mixes, resulted in marked improvement in workability and concrete placement, without any appreciable sacrifice of strength or other desirable properties.

Control methods. One of the factors instrumental in unifying field concreting practice was a pocket-size loose-leaf booklet of "Instructions for Concrete Inspection and Control" which was prepared and blueprinted locally to serve as an auxiliary guide in the application and interpretation of the contract specifications and was placed in the hands of each concrete inspector and other employee directly connected with the work. Chapters in the booklet were devoted to responsibilities and general duties of the inspector, inspection of materials, distribution of splices in longitudinal reinforcing bars, forms and their removal, methods and periods of curing, mixing and placing of concrete, making the slump test, measuring concrete materials, determination of moisture in sand or gravel, field calculations for converting designed mix to field mix, making and storing 6 x 12 in. test cylinders, sampling aggregates and making sieve analyses, and effects of proper and improper curing of concrete. Tables were also included giving bulking allowances for moist sand (for use in volumetric proportioning), designed mixes and their uses, batch weights (including moisture) per sack of cement for each approved mix, surface moisture in sand and gravel per sack of cement, relation between dry rodded and dry loose mixes, and other miscellaneous data for reducing field computations to the minimum. The latest revision of "Concrete Mixes—and Their Uses" is given in Table 1.

While the booklet of instructions served as a vehicle for arousing the inspector's interest, enlarging his knowledge of concrete, and supplying him with a set of working rules and tables, it could not be expected to maintain that spirit of alertness and constant attention to details so essential to the control of the numerous operations involved in the making of good concrete—especially when the work was progressing at top speed. In other

TABLE 1—CONCRETE MIXES AND THEIR USES

Mix (Dry Rodded)	Size of Gravel Inches	Admixture Per Cent	Approximate Slump Inches	Approximate W/C Ratio	*Sacks of Cement per Cu. Yd.	Work in Which Used
1:1.6:2.4	½	0	5–6	.70	7.11	Lock joint pipe.
1:1.7:2.8	1¼	0	6	.67	6.55	Siphon barrels (over 100-feet head).
1:2.0:3.25	1¼	1½	6	.81	5.75	Siphon barrels (under 100-ft. head) and siphon transitions.
1:2.4:3.6	1¼	3	2	.87	5.03	Canal lining (unformed).
1:2.4:3.6	1¼	3	4–6	.93	5.02	Tunnel lining, canal lining (formed), part of dam where 3-in. aggregate is not suitable, and ordinary thin-walled structures
1:2.75:6.25	3	3	2–3	1.10	3.42	Interior of main body of dam.
1½:2.75:6.25	3	3	3–5	.80	4.96	3-ft. exterior of main body of dam.

NOTE: Sacks of cement per cubic yard of concrete, slump, and w/c ratio values are the results of Ellensburg laboratory tests.

*Increasd by 5 per cent for field application.

words, a follow-up system was necessary and was provided by the following means:

(a) The various concreting jobs in progress were constantly visited by a so-called "Chief Inspector," a construction man of wide experience and mature judgment, who passed on the various features of the work and particularly those of a practical nature, such as the sufficiency of the forms, the proper placement and splicing of reinforcing bars, and the most suitable consistency of concrete for the particular conditions to be met. By consulting with the contractor, the inspector, and the division engineer, the Chief Inspector was able to check improper practices at their sources and iron out many differences or controversies.

(b) The Construction Engineer made a regular practice of going over all or part of the work at least once a week during the period of activity.

(c) Inspectors were required to make 6 x 12-in. test cylinders in accordance with a definite procedure and schedule, and to see that they were properly cared for until collected by a truck sent out from the project laboratory. After being collected, the cylinders were stored in moist-sand curing vats up to the date of breaking.

(d) Each concrete inspector was also required to prepare a daily "Inspector's Report" and "Cylinder Report," on mimeographed forms, principally to keep the project office informed and provide a permanent record for future reference. The "Inspector's Report" embraced all the essential features of the work, including location, work performed, delays, weather conditions, details of the mix and consistency, etc., and provided opportunity for the inspector to state his personal views or air his particular troubles. The reports were promptly checked by the division office engineer, then referred to the division engineer and mailed to the project office, where they were carefully reviewed. If any irregularities of consequence or requests for advice or assistance were noted, the matters were generally handled by correspondence, using short stereotyped-form of letters, or by a trip to the field on the part of the man directing the laboratory operations. The same agencies were utilized in the event any breaks of the field test cylinders proved to be abnormal and their abnormality was unaccounted for.

(e) Additional field trips were made by the laboratory representative, at rather frequent intervals and without warning, to check up on the more technical details incident to securing the specified concrete mixes (such as proper measurement of aggregates, water, etc.) or to investigate the conditions on particular jobs where there was reason to believe, based on information emanating from the Construction Engineer, Chief Inspector, or other source, that the control system was not functioning as well as it might.

Concrete mix used and basis for selection. The concrete mix used on the greater portion of the canal lining work was a 1:2.4:3.6 by volume, dry rodded, with 3 per cent diatomaceous silica admixture, and 2-in. average slump. A 1:2:4 mix was first attempted but proved deficient in sand for the best workability

and counteracting the harshening effect of field segregation of the coarse aggregate. Another change made in the early stage of construction was the reduction of the maximum size of the gravel from 1½ in. to 1¼ in.

Impermeability, rather than strength, was the governing factor in the selection of the canal lining mix. While strength reflects many of the desirable properties of concrete and serves as a convenient general measure of value, it is not necessarily a criterion of watertightness. In the case of the canal lining, the latter quality was regarded as of greater import in lengthening the life of the concrete by rendering it more resistant to the disintegrating forces to which the lining will be subjected, including contact with flowing water during the irrigation season and exposure to severe frost and variable moisture conditions during the balance of the year. What little test data on impermeability, for the particular aggregates chosen, was available at the time it became necessary to determine the canal lining mix, indicated that a mix leaner than a one to six by volume could not be relied upon to furnish the desired degree of watertightness, notwithstanding the relatively high compressive strength (about 4,500 pounds per square inch) corresponding to the one-six mix. This conclusion was substantiated by laboratory tests later conducted to investigate the water-pressure-resisting properties of 4 x 12 in. concrete discs of various mixes, as previously mentioned. Water pressures up to 500 pounds per square inch were employed in the tests, principally in connection with the determination of suitable mixes for siphons designed to operate under unusually high heads. Fig. 20 shows the latest form of impermeability apparatus developed. It is without the scope of this article to describe the details of the apparatus or give the results obtained from the tests, except to refer to one feature bearing directly on the canal lining mix; namely, that the addition of diatomaceous silica to the extent of 3 per cent by weight of the cement brought about a marked improvement in the watertightness of the concrete, although the original purpose in employing the admixture in the leaner mixes was mainly to improve the workability and lessen the tendency for segregation.

Measurement of concrete ingredients. A careful measurement of the ingredients in each batch of concrete for canal lining was

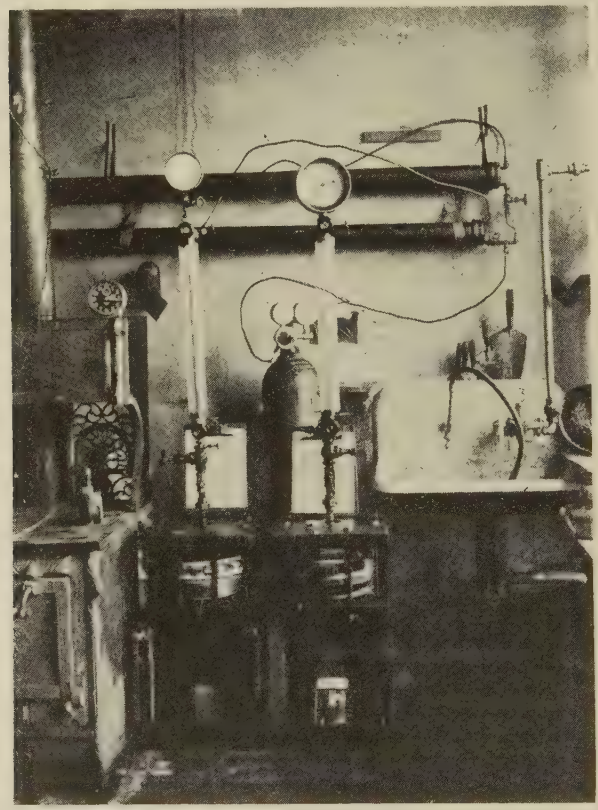


FIG. 20—IMPROVED TYPE OF APPARATUS FOR TESTING IMPERMEABILITY OF CONCRETE DISCS UNDER WATER PRESSURES OF 0-500 POUNDS PER SQUARE INCH

insisted upon. In all cases the sand was weighed and thus the problem of sand bulking was eliminated. The gravel was measured by weight or by volume, based on and occasionally checked by weight. Moisture in the aggregates, for determining the correct batch weights and accounting for the total water entering the concrete, was measured by means of apparatus embodying the water displacement principle, or by ordinary heating or alcohol-burning methods. The introduction of water at the mixer was gaged by the mixer man to bring the concrete to

the specified consistency or slump, the general practice being to regulate the mixer tank adjustment for the minimum water requirement and then add what little additional water was needed. This procedure was found to be the most practicable, and while on first thought it might seem to involve a material departure from rigid water control, a little reflection will reveal the fact that, with reasonably accurate measurement of the materials other than water, any variation in the water-cement ratio in bringing the concrete to the same consistency would be largely confined to the effect of variation in grading of the aggregates and would generally be insignificant. Considerable trouble was experienced by inspectors in determining the average amount of water added at the mixer, per batch of concrete, due principally to inefficient water regulating and measuring equipment on the mixers in use. In most cases the difficulties were wholly or partially overcome by calibrating the mixer tanks and supplementing the results with occasional check measurements of the mixer discharge by diverting it into a portable measuring tank.

Importance of controlling concrete consistency. Fortunately the contractors were as vitally interested as the Government in securing controlled concrete for the canal lining, inasmuch as consistency was a critical factor in the placement of the concrete on the side slopes. Even a variation of but 1 inch in slump made a marked difference in the results. If the slump was less than 2 in., the moving form left a honey-combed concrete; whereas a 3-in. slump produced a bulging and waving of the concrete. Best results were obtained with a slump of 2 to 2½ ins., a range which was quite consistently maintained by the mixer men after a little experience.

Field test data. The results of the 28-day breaks of field test cylinders on canal lining, for the 1928 construction season, are summarized in Table 2. Included in the table are average compressive strengths for the separate lining jobs and the work as a whole, a comparison between actual and predicted strengths, and data revealing the strength variations, including an improvised "Control Factor." Table 3 gives the compressive strengths of two series of 3 x 3 x 6 in. concrete test blocks which were cast

TABLE 2.—RESULTS OF 6 x 12-INCH TEST CYLINDERS—CANAL LINING—1928

Description	Contractor					Average of All Field Specimens (Moist Sand Cured)	Predicted Strength Based on Laboratory Control Curve
	C. F. Graff	General Construction Co.	S. H. Newell & Co.	Derbon Constr. Co.			
				Moist Sand Cured	Surface Treatment Cure		
Number of field test specimens	59	91	72	155	90	377	
Average reported W/C ratio	.77	.82	.83	.84	.84	.82	*.82
Average 28-day compr. strength	4359	4914	4572	4466	3755	4562	4400
Mean variation in strength was as follows:							
A. Per cent within 10 per cent of average strength	65%	62%	53%	51%	47%	56%	
B. Per cent within 15 per cent of average strength	87%	82%	68%	74%	74%	77%	
C. Per cent within 20 per cent of average strength	90%	98%	92%	85%	87%	90%	
Control factor = $\frac{A+B+C}{3}$	81%	81%	71%	70%	69%	74%	

*Water-cement ratio given represents average of field values reported and corresponds to laboratory water-cement ratio of about .87, the difference of .05 being due to aggregate absorption estimated at .5 per cent by weight.

TABLE 3.—COMPRESSIVE STRENGTH OF 3-IN. BY 3-IN. BY 6-IN. CONCRETE BLOCKS FROM CANAL LINING

Age at Test	From General Construction Company lining (Continuous sprinkling for 14 days)				From Derbon Construction Company lining (Cured by asphalt paint application—Hunt Process)			
	S. Slope	Floor	N. Slope	Average	S. Slope	Floor	N. Slope	Average
7 Days	3220	3610	3610	3480	4920	5305	4055	4760
14 Days	3780	4610	4000	4130	4330	5055	4330	4572
28 Days	4740	5000	4535	4758	5480	4000	4040	4507

NOTE: Lining containing blocks poured on July 20, 1928. Followed by 8 days of extremely hot weather, during which the temperature mounted to 110 degrees Fahrenheit and exceeded 100 degrees on 6 successive days.

and cured simultaneously as an integral part of the canal lining. One series is representative of water-cured lining, and the second of lining cured by the asphalt paint application (Hunt process). A striking feature of the test results in Table No. 3 is the gradual increase in strength of the water-cured specimens, as compared with the rapid hardening of the surface-treated blocks. As stated in the explanatory note, the block tests were carried on during a period of extreme summer temperatures. The slower-hardening of the water-cured blocks is accounted for mainly by the cooling effect of the continuous sprinkling.

Value of concrete control. Judging from the opinions expressed, there is little doubt in the minds of those connected with the concrete work, or of even the casual observers who witnessed the concreting operations or viewed the completed work, that the effect of the more systematic and precise concreting methods practiced was noticeably reflected in the finished product. (Figs. 21 and 22). The only question which remains, apparently, is whether the improvement realized will manifest itself in the form of decreased maintenance and increased life of the concrete to a sufficient extent to compensate for the additional effort expended, and while there is a general feeling of assurance that it will, the proof of the pudding is in the eating and time alone will tell. From the experience gained on the Kittitas division, this much at least may be said for control: (1) That it shows unmistakable evidence of improved concrete and may yield big financial returns; (2) That it brings forcefully to the attention of inspectors, and others engaged in directing the operations, the many influences affecting concrete in the making and the necessity for coordinating and controlling the various detail processes if concrete of uniformly high quality is to be produced; (3) That when control embodies properly made tests of laboratory and field specimens, it provides a definite and intelligent means for establishing the field mixtures and gauging the qualities of the final product, even though such specimens may not be truly representative of the concrete in the work; and (4) That it stimulates an interest in concrete and inspires further investigation, which in turn should eventually rid the manufacture of concrete of its major perplexities.



FIG. 21—LOOKING DOWN CONCRETE LINED SECTION OF MAIN CANAL FROM OUTLET OF PETERSON CREEK SIPHON. NOTE SAFETY LADDER ON RIGHT HAND SIDE SLOPE. SAFETY LADDERS PLACED EVERY 250 FT. ON ALTERNATE SIDE SLOPES IN LINED SECTIONS OF THE CANAL



FIG. 22—ANOTHER SECTION OF CONCRETE LINED CANAL. LOOKING DOWN THE MAIN CANAL NEAR EASTON, WASHINGTON. NOTE TRUE ALIGNMENT OBTAINED BY USE OF MOVABLE CONCRETE PLACING PLATFORM

Personnel. The construction of the Kittitas division of the Yakima project was under the direction of Construction Engineer Walker R. Young. The development of technical control procedure for the concrete work and the prosecution of the laboratory work were under the general direction of Office Engineer Arthur Ruettgers. Division Engineer A. A. Whitmore was in direct charge of the reach of Main Canal which included the lining work contracted to C. F. Graff, S. H. Newell & Co., and the General Construction Co. The reach of canal comprising the lining constructed by the Derbon Construction Co. was in charge of Division Engineer V. W. Russell. The principal designing work for the Bureau of Reclamation is done in the Denver office under the general supervision of J. L. Savage, Chief Designing Engineer. All engineering and construction work is under the general supervision of R. F. Walter, Chief Engineer, with headquarters at Denver, and all activities of the bureau are under the general charge of Dr. Elwood Mead, Commissioner, with headquarters at Washington, D. C.

Readers are referred to the JOURNAL *for February, 1931, for discussion which may develop. Such discussion should reach the Secretary by January 1, 1931.*

Construction and Design Features of Haydite Concrete.

BY F. E. RICHART* AND V. P. JENSEN†

INTRODUCTION

General Statements: The development of the use of reinforced Haydite concrete has introduced various problems in design and construction with this material. While light weight is the property that makes desirable the use of Haydite concrete, the engineer and contractor require information on other properties, particularly those affecting strength, durability and ease of construction. There is a demand for information on proportions to produce a specified strength and workability, and on methods of quality control. The designer wants, in addition to data on unit weight and strength, information as to the elastic behavior of the material and its action in a structure. The effect of the relatively low modulus of elasticity of Haydite concrete is of importance, and may have a considerable influence upon the economy of design.

A recent paper by the authors, reporting an investigation of Haydite concrete in progress at the University of Illinois, gave test data on compressive strengths, bond and diagonal tension resistance, together with various other strength properties of Haydite concrete.‡ An attempt will be made in this paper to analyze certain of these test data and to apply the results to design and construction work.

The investigation mentioned above is being made at the Materials Testing Laboratory, University of Illinois, in cooperation with the Western Brick Co., Danville, Illinois, manufacturers of "Western" Haydite aggregates.

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†Research Assistant in Theoretical and Applied Mechanics, University of Illinois.
‡Proc. A. S. T. M., Vol. 30, Part II, 1930. "Tests of Plain and Reinforced Haydite Concrete."

The work is being conducted by the authors under the administrative direction of Dean M. S. Ketchum, Director of the Engineering Experiment Station, and Professor M. L. Enger, Head of the Department of Theoretical and Applied Mechanics, University of Illinois. I. N. Doughty, Manager of Research, and Frank Payne, Engineer, of the Western Brick Co., have represented their organization as an advisory committee on the work.

PROPERTIES OF MATERIALS

Aggregates: The Haydite aggregates to which this discussion applies are the "Western" fine or "A" size material, passing a $\frac{3}{16}$-in. round hole screen, and the coarse or "C" size, retained on the $\frac{7}{32}$-in. screen and passing the $\frac{3}{4}$-in. screen. The range in values of the properties of a number of lots used are given in Table 1. The apparent specific gravity of material passing the 200-mesh sieve was found to be 2.56, indicating that the absolute specific gravity of the material is as high or higher. The coarse particles, with an apparent specific gravity of 1.25 to 1.32, evidently have about 50 per cent porosity. The apparent specific gravity of the material increases as the size of particle decreases. There are usually a few particles in a sample that will float in water. The specific gravity of a graded sample is seen to be the average of individual values that vary considerably and is not so definite a property as that of a homogeneous material.

Types of Concrete Mixture: Reference will be made frequently to two types of mixture, one made with a natural sand and coarse "C" Haydite, which will be denoted as "C-Haydite" concrete; the other made with fine "A" Haydite and coarse "C" Haydite, denoted as "All-Haydite" concrete. The sand used in the first mixture was a torpedo sand from Covington, Indiana, having a fineness modulus of 3.08, unit weight, loose, dry, of 106 lb. per. cu. ft., absorption of 1.0 per cent, and specific gravity of 2.67.

Absorptive Properties of Aggregates: The absorption of mixing water, usually a minor factor in the production of concrete with sand, gravel and stone aggregates, becomes a matter of real importance when the aggregates are capable of absorbing a large amount of water. This presents a two-fold problem, of which one part concerns the drying of the concrete and consequent

change in workability due to rapid absorption of water by the Haydite during and immediately after mixing; the other involves the determination of the true water-cement ratio, or some related property, which varies as the absorption of the mixing water proceeds. The latter variation obviously adds some difficulty to the use of water-cement ratio as a strength control.

The tests of absorption made thus far have been of a preliminary sort; however, a more thorough study is now being started and should furnish needed data. The tests indicated one unexpected phenomenon; the fine Haydite absorbed a larger percentage of water by weight in a given length of time than did the coarse material. Considering that the material consists of a mass of closed cells of infinitesimal size, it is seen that in view of the greater surface area of the fine material, a certain depth of penetration by the water should produce the greater absorption in the fine material. It is probable that the total absorption over a long period would be greater for the coarse material; tests up to 7 days showed continuing absorption at the end of the period. It is also likely that the absorption by the very fine material, such as that passing the No. 100 sieve, would be small due to the lower porosity.

The absorption tests made were on dry material and on material with various initial moisture contents. The procedure used with the coarse aggregate was the usual immersion method, using a covered wire basket suspended from one arm of a balance. The material was surface dried by rolling it on a paper towel. The method finally used with the fine material was to place a sample of the soaked material in a centrifuge, which was used to remove surface water. The criterion for surface dryness was the ability of the material to run freely through the fingers. In both tests the amount of absorbed water was found by oven-drying.

Fig. 1 (a) shows the absorption determined for dry fine and coarse Haydite for various periods of immersion. Fig. 1 (b) shows similar data for both dry and initially moist materials for various immersion periods. It is clear that Haydite does not have a *fixed* amount of absorption in a given time, as usually assumed for ordinary aggregates, but the amount depends upon the initial moisture content. Moist aggregates show a larger total absorption than initially dry ones, especially in the case of

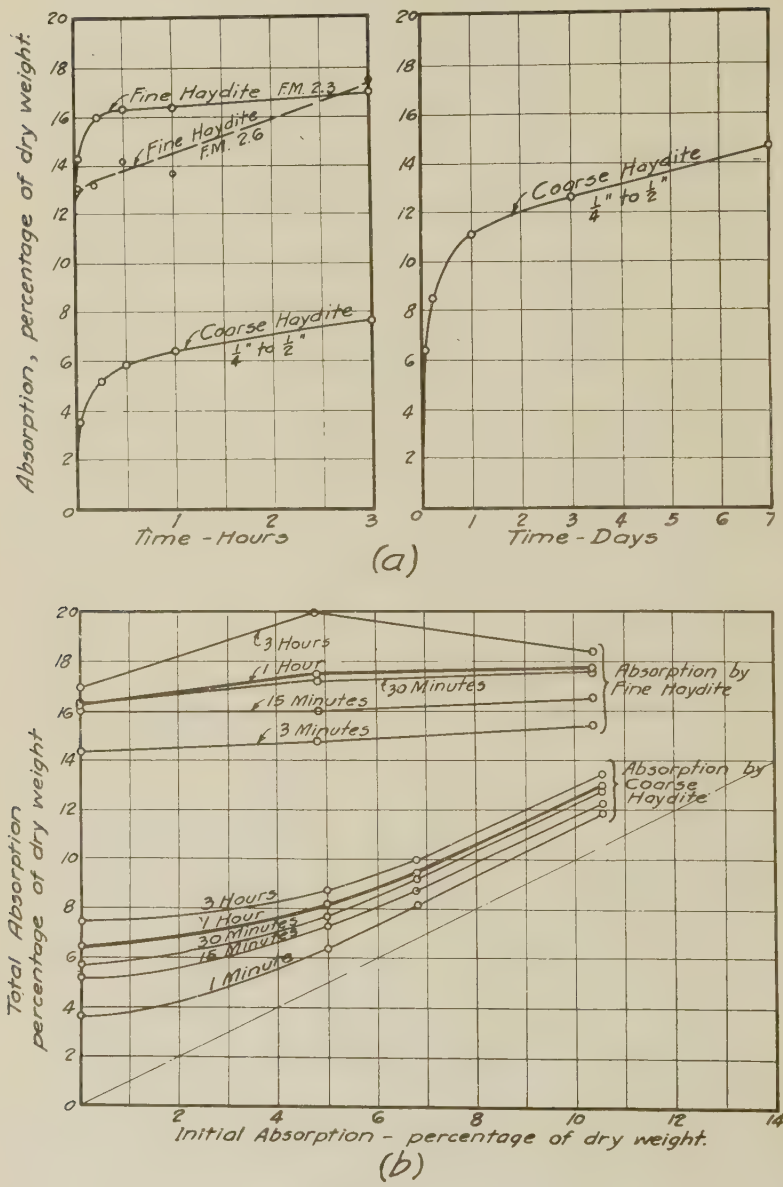


FIG. 1—(a) ABSORPTION OF WATER BY INITIALLY DRY HAYDITE AGGREGATES FOR VARIOUS PERIODS OF IMMERSION. (b) ABSORPTION BY HAYDITES HAVING VARYING INITIAL MOISTURE CONTENTS

the coarse material. The fine material reaches its absorptive capacity more quickly than the coarse and is less affected by initial moisture conditions.

The effect of a variation in water-cement ratio due to error in absorption allowance may be seen from an example. In a one-bag batch of a 1:2:3 mixture, there would be roughly 245 lb. of Haydite. An error of 2 per cent by weight of aggregate in the absorption allowance would change the amount of free water by 4.9 lb. and the value of w/c by $\frac{4.9}{62.4} = 0.08$. With a water-cement ratio of 1.0 this will cause a variation in strength of 15 per cent or more.

There is little information regarding the period of time that aggregates should be immersed in water to produce an absorption equal to that taking place during the mixing and placing of concrete. Periods varying from 15 minutes to 3 hours have been assumed by various investigators, without great difference in result when ordinary aggregates were used. Observation of fresh concrete in forms indicates that insofar as water content fixes the voids in the concrete, the mass takes its final volume (neglecting subsequent shrinkage of the order of 0.1 to 0.2 per cent of volume) within 15 minutes after the concrete is placed. Changes in water-cement ratio due to further absorption by the aggregates may affect the process of hydration and thus affect the strength. An estimate of this effect has led to the use in this investigation of an absorption allowance corresponding to a one-hour immersion test. Values for one-hour absorption for material with various initial moisture contents are shown by the heavy curves of Fig. 1 (b).

It will be noted that Fig. 1 (b) refers to initial absorbed moisture, not total moisture content. The proportions of absorbed and free moisture in a sample will depend entirely upon how the moisture content has been taken on. If a wet sample is drying out the amount of absorbed moisture will be high; a sample just wet down will contain mostly free surface water.

CONSTRUCTION FEATURES

Relations Between Water-Cement Ratio and Strength: In a study of the relation between water-cement ratio and strength

for Haydite concrete, several series of cylinder tests were made, including mixtures with sand and gravel aggregates, sand and coarse Haydite, and fine and coarse Haydite. These were made of several proportions and consistencies and tested at ages of 7 and 28 days. Two groups were made with fine and coarse Haydite, one with oven-dried material, the other with moisture contents of 6 per cent in the coarse and 27 per cent in the fine material. In determining the water-cement ratios, absorption allowances have been used as follows: for sand and gravel, 1 per cent by weight; coarse Haydite, dry, 7 per cent, moist, 9 per cent; fine Haydite, dry 14 per cent, moist, 20 per cent. Data from the 28-day tests are plotted in Fig. 2. All of the points fall within a fairly narrow zone on the diagram, though there are definite differences in the average curves for the four series of tests. The All-Haydite concrete made with moist aggregates appears to be consistently stronger than that made with dry Haydite, and as strong as that made with sand and gravel. The difference in the Haydite concretes may be due to a better bond between the moist particles and the cement paste than that obtained with the dry particles, to differences in absorption not fully accounted for in the corrections applied or to differences in the water-cement ratio relation. All of these curves give strengths considerably above the values recommended in the Joint Standard Building Code of the American Concrete Institute for use with average materials. These rather high strengths call attention to the cement used, which was a new lot of standard Universal cement, passing the Standard Specifications for Cement current when the tests were made. Tensile strengths of standard Ottawa mortar briquets were: at the age of 7 days, 258 lb. per sq. in.; at 28 days, 370 lb. per sq. in.

It is recommended that in the design of Haydite concrete when very careful absorption determinations are not made and the work is done under ordinary conditions of supervision, somewhat lower strengths be assumed for a given water-cement ratio than those given in Fig. 2. The values recommended in the Joint Standard Building Code noted above, which are probably sufficiently conservative, are as follows:

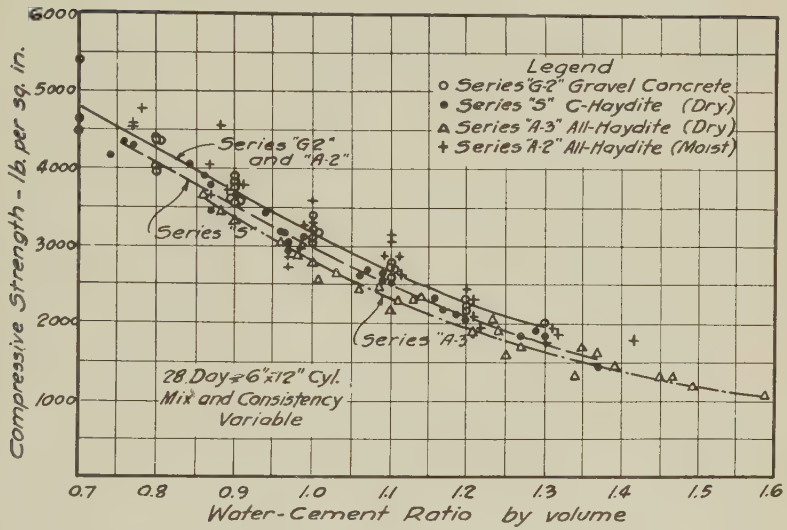


FIG. 2—RELATION BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH FOR THREE TYPES OF CONCRETE

Water-Cement Ratio		Compressive Strength* lb. per sq. in.
By Volume	Gallons of Water per Sack of Cement	
1.10	8¼	1500
1.00	7½	2000
0.90	6¾	2500
0.80	6	3000

*From tests of 6 by 12 in. cylinders, moist cured for 28 days at 70° F., tested moist.

The indication in Fig. 2 that greater strength is obtained through the use of moist Haydite aggregates than with dry ones, even when a logical correction is made for differences in absorption, is borne out by several other groups of tests. This favors the practice of pre-wetting of aggregates in the stock-pile previous to placing them in the mixer, which has been done, however, for the primary purpose of securing better and more uniform workability and to prevent undue drying out of the concrete after

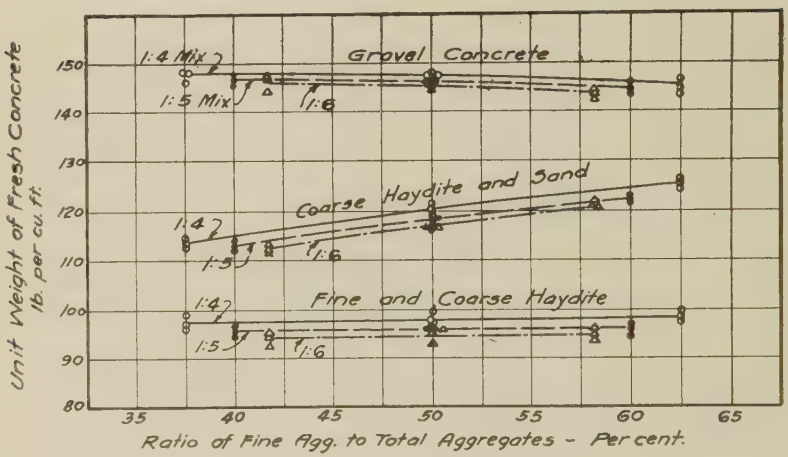


FIG. 3—UNIT WEIGHT OF GRAVEL AND HAYDITE CONCRETES

it was placed. Such pre-wetting, unless very systematically done, may increase the difficulty of applying proper absorption corrections where a close control of strength is required.

Proportions and Workability. Some users of Haydite aggregates have run into difficulty due to an attempt to use as large a proportion of coarse Haydite as would be used with 1½-in. gravel. It has been established by experience and tests that the amount of coarse aggregate usable decreases as the size of particle decreases, at least below the 1-in. size. While an aggregate of ½-in. size may contain the same percentage of voids as one of 1½-in. size, it is obvious that the size of the voids is smaller in the ½-in. material and will not accommodate sand grains without separation of the coarse aggregate particles. The proportion of coarse to total aggregates recommended by the Portland Cement Association in "Design and Control of Concrete Mixtures," 3rd. Ed. 1929, is 0.50 to 0.70 for coarse aggregate of maximum size of 1-in. or over; 0.40 to 0.60 for a maximum size of ¾-in.; and 0.30 to 0.45 for a maximum size of ⅜-in. Proportions of "C" Haydite (which is composed of material passing a ¾-in. round hole screen) varying generally from 40 to 60 per cent of the total were used in the tests represented by Fig. 2. Considering the resulting

workability obtained with the use of an excess of fine aggregate and also the accompanying increase in weight of the sand and "C" Haydite mixture, it is believed that a mixture of 45 per cent fine and 55 per cent coarse is the most desirable combination, at least with the sand and Haydite mixtures. For All-Haydite concretes it may be just as economical to use equal parts of fine and coarse material.

Data of C-Haydite and All-Haydite mixtures (see Series S and A-3 of Fig. 2) are given in Table 2 as a guide to proportions and accompanying slumps usable with various water-ratios. The materials used were in all cases initially dry. If moist aggregates are to be used, bulking and moisture content tests should be made, so that volumes of moist material containing solids equal to those of the dry materials may be used. The bulking effect is small for the coarse Haydite and usually will not exceed 5 or 6 per cent, but for the fine Haydite the bulking may be important. Thus the fine Haydite weighing 55 lb. per cu. ft. by dry loose measure, weighs 51 lb. per cu. ft. with 27 per cent of moisture present. This 51 lb. consists of 40.1 lb. of dry Haydite and 10.9 lb. of water. To obtain 55 lb. of dry Haydite in the mixture requires $\frac{55}{40.1}$ or 1.37 cu. ft. of the moist material. With this 37 per cent bulking in the fine aggregate and assuming a 5 per cent bulking in the coarse, for a mix of 1:2:3 concrete as listed in Table 2, the volume proportions with the moist materials should be 1:2.74:3.15. Similarly, the quantities required per cubic yard of concrete as given in Table 2 would be multiplied by the ratios of 1.37 and 1.05 for the fine and coarse materials, respectively.

The use of moist aggregates just described will naturally affect the water requirements. In a one-bag batch the 2.74 cu. ft. of fine aggregate contain 10.9 lb. of water per cu. ft. or 29.9 lb. This must be considered in computing the amount of water for mixing and absorption requirements.

The quantities of materials in Table 2 contain no allowance for waste or for settlement of material in forms. As explained, the aggregate quantities must be increased for bulking when moist aggregates are used. If these proportions are used without

correction for bulking, a richer and stronger mix will result, and all quantities of materials, including the cement, will be increased. The amount of such increase, in per cent, is roughly equal to the per cent bulking of the fine aggregate times the ratio of fine to total aggregates.

Table 2 shows a range of workabilities as shown by values of slump from 1 to 10 inches. For making test beams with fairly heavy reinforcement, very satisfactory placing of the concrete was accomplished with slumps of 6 and 7 inches. Good surfaces were obtained next to the forms and there was practically no honey-combing or segregation of the materials. For slabs and large beams slumps of 4 to 6 inches should give satisfactory workability.

Unit Weight of Concrete: Data on unit weight of freshly placed concrete as found from test cylinders are given in Fig. 3. It is seen that the weights of gravel and All-Haydite concretes do not vary greatly with changes in proportions and consistencies corresponding to those of Table 2. The values shown in Fig. 3 are for concrete made with dry aggregates. The weight of All-Haydite concrete made with moist aggregates was 3 to 6 lb. per cu. ft. greater than that of Fig. 3, which is further evidence that the total absorption of moist aggregate is greater than for that initially dry. The increase in weight of "C"-Haydite concrete with increasing amounts of natural sand is self-explanatory.

DESIGN CONSIDERATIONS

General Considerations: Of the several phases of concrete design those of primary importance are—first, the selection of a concrete strength, second, the determination of the necessary and usable proportions of materials, and third, the actual design of the structural members. Data pertaining to the first and second phases have been presented and further discussion of economy as it relates to them would necessarily involve relative prices of materials, relative quantities of the different materials which could be used to produce a desired strength and consistency of concrete, and proportional costs of raw materials and concrete in place. Current local prices and the needs of individual jobs will govern the comparisons to be made and therefore this section

will be devoted entirely to the factors which affect the design of the structural members. These factors are lightness in weight and low modulus of elasticity.

In the following analysis the symbols and notation are those in general use, as defined, for example, in the A. C. I. Joint Standard Building Code, Section 602. In addition, use has been made of the relation *Resisting Moment* = Kbd^2 for balanced reinforcement, also the subscript g has been used with terms referring to gravel concrete.

It may be noted that the comparisons to follow involve values of strength and modulus of elasticity from tests of cylinders rather than of beams. The limited amount of beam test data available are in accord with the analyses given; Haydite concrete beams show larger values of k and consequent smaller values of j, and increased strains and deflections at a given load, as compared to gravel concrete beams. Haydite concrete beams tested in studies of diagonal tension showed no initial failures at the extreme fibre in compression, even though percentages of longitudinal reinforcement (structural grade) as great as 2.8 were used, and tensile failure of the steel was developed. Obviously, beam and slab tests corresponding to the range of designs considered here would be impracticable, and such tests have not been made. Instead, the authors have felt justified in determining properties of the material and in applying the results by means of well-established theory of the behavior of such members.

Effect of Lightness in Weight: Unit weights of concrete made of gravel and sand, coarse Haydite and sand, and of fine and coarse Haydite are shown in Fig. 3. For ordinary mixtures "C"-Haydite concrete will weigh 80 per cent, and All-Haydite concrete 67 per cent, of the weight of gravel concrete. The effect of a reduction in dead load depends upon the percentage of dead to total load and upon the manner in which the designer may take advantage of the reduced load. His beam sizes, spans, or slab thicknesses may be limited by considerations other than stress, in which case it may not be possible to obtain full advantage.

Comparison has been made in Table 3 of the three kinds of concrete (gravel, "C"-Haydite, and All-Haydite) for ratios of dead load to total load varying from 60 to 30 per cent, upon the conditions that the resisting moment which must be developed is

proportional to the load per sq. ft. of floor area, and that b and d for slab or beam are the same for all three kinds of concrete. The figures shown are ratios of total loads to be carried and are equal to the ratios of resisting moments to be developed when the span is the same for all three concretes. Values of j for "C"-Haydite and All-Haydite concretes with balanced reinforcement are shown in the following section to be approximately 97 and 95 per cent respectively of j for gravel concrete. Haydite concretes are shown to require higher percentages of steel for balanced reinforcement as a consequence of higher values of n. Therefore in the comparison of Table 3, which imposes constant span and depth of slab, a decreased bending moment due to decreased load results in a lowering of the percentage of steel required for the Haydite slabs and brings the values of j for Haydites nearer to the value for gravel than is indicated by the proportions of 97 and 95 per cent. The relative percentages of steel required for the three concretes are thus shown to be approximately equal to the values given in Table 3 of relative percentages of resisting moment which must be developed.

The effect of taking advantage of the reduced weight of Haydite concrete by reducing the depth of the slab is to produce an additional secondary reduction in the values given in Table 3 of from 1 to 4 per cent, the higher percentage being for the greatest percentage of dead to live load. In a comparative design of gravel and Haydite concrete in which d is varied and balanced reinforcement is maintained the additional effects of concrete strength and variation of E enter the problem. These effects are discussed separately in the following section.

The percentage of reduction in column loads to be carried when Haydite concrete is used in place of gravel concrete is somewhat greater than those indicated for slabs in Table 3, due to the reduction factor which is applied to the live loads under many building codes. The effect of such a live load reduction factor becomes greater as the number of stories between the column to be considered and the roof is increased. Since the column is designed for the remaining live load and the dead load, a decrease in the latter becomes quite important.

A considerable economy may result from dead load reduction under certain conditions of design due to the ability to use a cheaper type of foundation than would have been required for the heavier dead load. Complete data concerning the individual job would necessarily have to be available for a determination of this possibility.

Modulus of Elasticity of Haydite Concretes: Experimental values of initial moduli of elasticity of concretes made with gravel and sand, coarse Haydite and sand, and fine and coarse Haydite are shown in relation to compressive strength in Fig. 4 (a). Each value is the average of 3 tests. Mixes varied from 1:4 to 1:6, percentages of fine aggregate to total aggregate from 37.5 to 62.5, and slumps from 1 inch to 10 inches. It will be seen that for each kind of concrete the variation in E with strength may be represented by a straight line for strengths between 2000 and 4000 lb. per sq. in. It will also be noted that the moduli for "C"-Haydite and All-Haydite concretes average about 75 and 55 per cent, respectively, of the values for gravel concrete.

Fig. 4 (b) shows the relation between the moduli found by test and the values which are obtained by using the A. C. I. building code value of $1000f'_c$ for gravel concrete and values 75 and 55 per cent as great for "C"-Haydite and All-Haydite concretes, respectively. As another approximation, values of constant moduli of 3,400,000 for gravel, 2,500,000 for "C"-Haydite, and 1,900,000 for All-Haydite concrete are shown in the figure for comparison with the experimental and A. C. I. values. The three relations between E and strength will be used later in comparative studies.

For a more complete study of the possibilities of variation of secant moduli and to give information concerning relative deformations of the three concretes, the stress-strain curves of Fig. 5 are presented. These are typical curves taken from individual 6 by 12-in. cylinders of varying mix and consistency. Curves were obtained by means of an attached extensometer and a semi-autographic recording device. A constant ultimate unit deformation of about 0.0015 was found for gravel concrete of all strengths except those considerably below 2000 lb. per sq. in., this value being in agreement with numerous previous tests.

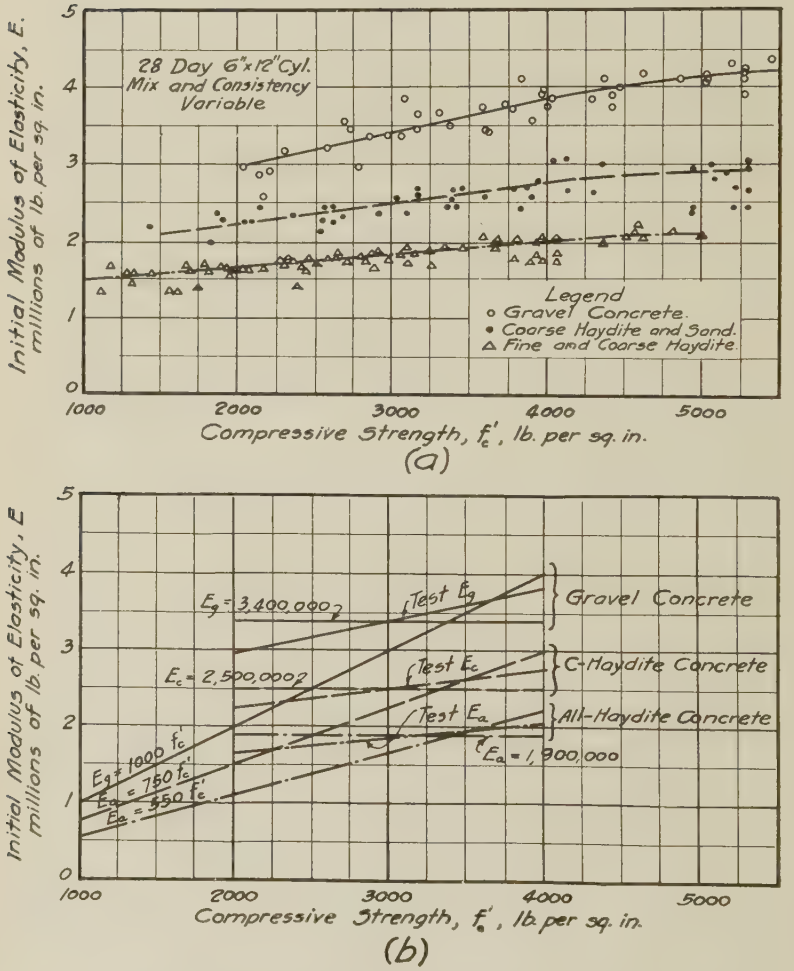


FIG. 4—RELATION BETWEEN INITIAL MODULUS OF ELASTICITY AND COMPRESSIVE STRENGTH FOR THREE TYPES OF CONCRETE

Ultimate unit deformations of the Haydite concretes are shown to be greater than those of gravel concrete, the proportional increase being greater for the concretes of higher strength. Deformation of All-Haydite concrete shows an increase over that of "C"-Haydite concrete similar to the increase in deformation of "C"-Haydite over that of gravel concrete. A consideration of the nearly straight stress-strain curve for high-strength gravel concrete will show that concretes having initial moduli 75 and 55 per cent as great must necessarily receive higher deformations in order to develop the same stress as the gravel concrete.

The curves of Fig. 5 have been replotted in Fig. 6 with the abscissas for the Haydite concrete reduced by the differences in horizontal intercepts of the tangents which represent the initial moduli of these concretes and gravel concrete. In other words, all curves have been brought approximately to a common tangent, that for an intermediate grade of gravel concrete. The curves as represented thus are scarcely more varied than is to be expected from a group of individual cylinders made with a single kind of aggregate. The similarity of these curves leads to the conclusion that for deformations up to 50 per cent of the ultimate deformation the relations of secant moduli of Haydite and gravel concretes are essentially the same as those of initial moduli. A further conclusion is that the primary difference in deformations is the difference in "elastic" deformation (deformation up to the line for initial modulus) as indicated by the ratios 75 and 55 per cent, and that the difference in "plastic" deformation (remaining deformation) is slight and of secondary importance. Since the "plastic" deformation gives an indication of the rate of breaking down of the structure it would seem that the Haydite particles themselves do not affect the rate of internal local failure and final complete rupture of plain concrete in compression.

Effect of Low Modulus of Elasticity on Design of Beams and Slabs: Comparative properties and design factors of rectangular beams and slabs have been computed and tabulated for gravel, "C"-Haydite and All-Haydite concretes having compressive strengths of 2000, 2500, 3000 and 3750 lb. per sq. in., using values of E from Fig. 4 (b). While a comparative design based upon the test values of E should probably give the truest indication of

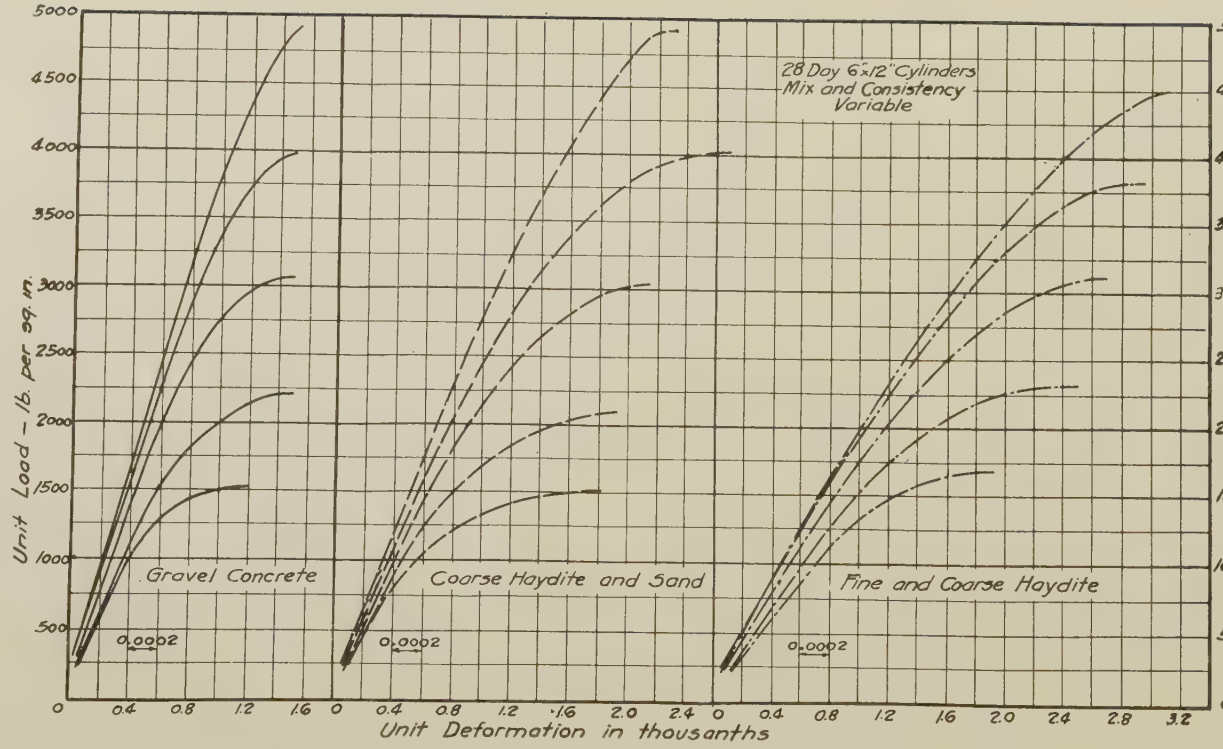


FIG. 5—STRESS-STRAIN CURVES FOR THREE TYPES OF CONCRETE

the differences in the concretes and their resulting behavior in flexure, the designer must necessarily use a codified value of E, and therefore the comparisons given in Tables 4, 5 and 6 are based on three different assumptions, respectively; (1) using test values of E, (2) using three proportions of the A. C. I. building code value of E, and (3) using constant value of E for each material. It is to be noted that the comparisons made in these tables are for the purpose of determining the effect of variation in E only and do not include the additional effect of lightness in weight in reducing the load to be carried and the section required.

In section A of Table 4, which is based upon experimental values of E, balanced reinforcement, $f_s = 20{,}000$ lb. per sq. in., and $f_c = 0.40\, f'_c$, the factors, n, k, j, p, and K are given for all three kinds of concrete. It will be seen that the lowered E and increased n for Haydite, while causing a slight decrease in j and a considerable increase in k and p, have the effect of increasing considerably the resisting moment factor K. The slab and rectangular beam comparisons which follow in the table are based upon various design limitations.

In section B, Table 4, under the heading "Slab Comparison Based Upon Constant Resisting Moment, b constant, d varies," are given figures which are intended to show the net effect of the changed design factors for Haydite concretes. These have been computed as follows:

Slab with Variable Depth, d.

With balanced reinforcement,

$$R.\ M.\ (\textit{Resisting Moment}) = Kbd^2 = 12\ Kd^2$$

$$d = \left(\frac{R.\ M.}{12\ K}\right)^{\frac{1}{2}}$$

For a constant resisting moment the ratio of depth, d, required for Haydite concrete slabs to depth, d_g, required for gravel concrete is,

$$\frac{d}{d_g} = \left(\frac{K_g}{K}\right)^{\frac{1}{2}}$$

Ratios of $\frac{d}{d_g}$ for "C"-Haydite concrete are shown in Table 4 to vary from 92 to 91 per cent and for All-Haydite concrete to

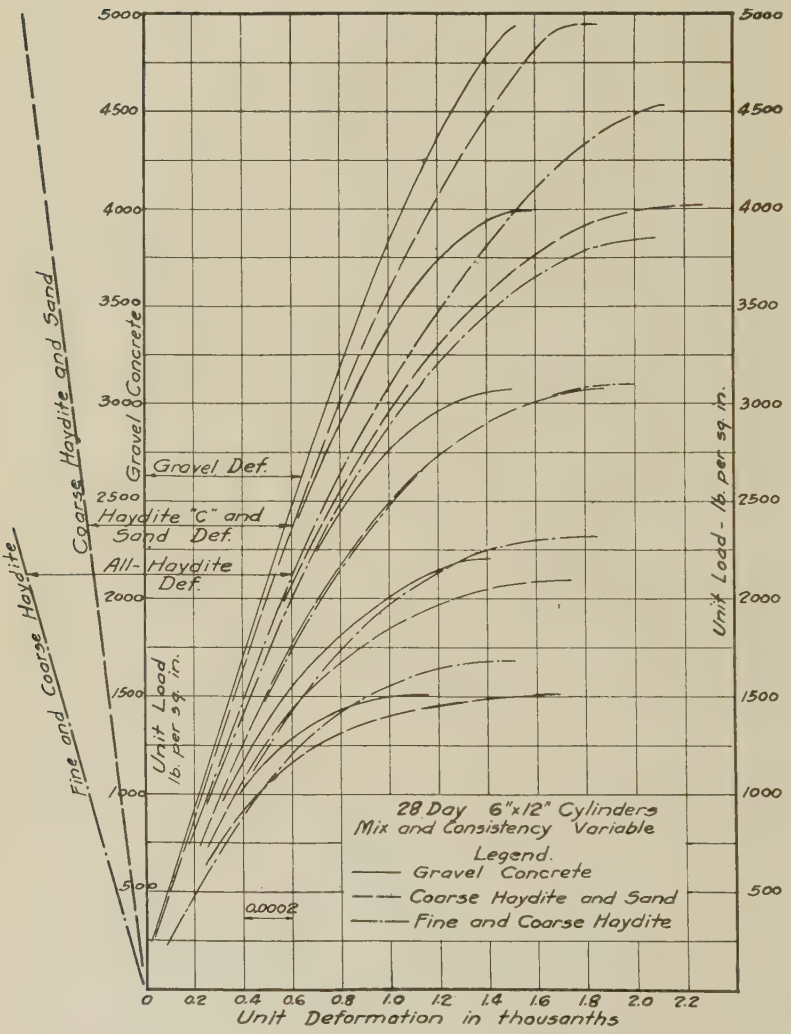


FIG. 6—MODIFIED STRESS-STRAIN CURVES FOR THREE TYPES OF CONCRETE

vary from 84.5 to 87.5 per cent. Percentages of total volume of concrete required would be from 1 to 3 per cent higher than these percentages of depth due to a constant thickness of covering.

Ratios of steel areas, A_s, required for Haydite slabs to steel areas, A_{sg}, required for gravel concrete are net ratios based on the increased p required for balanced reinforcement in Haydite and the reduced d as shown by the ratio $\frac{d}{d_g}$.

Thus:

$$\frac{A_s}{A_{sg}} = \frac{d}{d_g} \cdot \frac{p}{p_g}$$

Percentages of $\frac{A_s}{A_{sg}}$ (Table 4, part B) show for this comparison a requirement of from 111 to 110 per cent of steel area for "C"-Haydite concrete and from 123.5 to 121.5 per cent for All-Haydite concrete. The total effect is then to develop the same resisting moment with "C"-Haydite concrete with d reduced 8 to 9 per cent and A_s increased 11 to 10 per cent, and with All-Haydite concrete with d reduced 15.5 to 12.5 per cent and A_s increased 23.5 to 21.5 per cent.

Section C of Table 4, which is headed "Slab and Rectangular Beam Comparison Based upon Constant b and d for All Concretes," applies to the case in which a possible reduction in depth of slab is not utilized. It shows that with "C" Haydite concrete an increase of 18 to 17 per cent in resisting moment is obtained by the use of 21 to 20 per cent more steel, and that with All-Haydite concrete an increase of 39 to 31 per cent in resisting moment is obtained by an increase of 46 to 39 per cent in steel. The exact value of the increased carrying capacity or possibility of increased span length which is purchased in this case by the use of a higher percentage of steel is not at once easily determined as a simple percentage. If, however, no increase in resisting moment is desired, a constant resisting moment with constant depth of section may be provided with a Haydite concrete slab by the use of slightly more steel than that required by gravel concrete. This is due to the slight difference in j for the two materials, an effect due only to variations in modulus of elasticity. The com-

pensating effect of a reduction in weight in reducing the steel required with Haydite concrete is considered in Section 12.

In Section D of Table 4 comparisons of rectangular beams are based upon the condition that it is possible to reduce both b and d and that b and d for Haydite concrete beams are proportional to b and d for those of gravel concrete. Calculations were made as follows:

Rectangular Beams.

With balanced reinforcement,

$$\begin{aligned} R.\ M. &= Kbd^2 \\ b &= Constant \times d = Cd \\ bd &= Cd^2 \\ R.\ M. &= CKd^3 \\ d^2 &= \left(\frac{R.\ M.}{C\ K}\right)^{2/3} \end{aligned}$$

For a constant resisting moment,

$$d^2 = Constant \times \left(\frac{1}{K}\right)^{2/3}$$

$$\frac{bd}{(bd)_g} = \frac{d^2}{d_g^2} = \left(\frac{K_g}{K}\right)^{2/3}$$

These area ratios, $\frac{bd}{(bd)_g}$ for "C"-Haydite concrete are shown to vary from 89 to 90 per cent and for All-Haydite concrete to vary from 80 to 83.5 per cent of gravel areas.

Net ratios of steel areas, A_s, required for Haydites to areas, A_{sg} required for gravel concrete are 107 to 109 per cent for "C"-Haydite and 117 to 116 per cent for All-Haydite concretes. For these computations,

$$\frac{A_s}{A_{sg}} = \frac{bd}{(bd)_g} \cdot \frac{p}{p_g}$$

giving ratios of steel areas based on the reduced beam section as was done in the case of slabs of variable depth.

The comparative design ratios given in Table 5 using proportional A. C. I. Code values of E, and in Table 6 using constant values of E, were all computed in the manner described for Table 4.

It is to be noted that the comparative design ratios in Tables 5 and 6 do not differ greatly from those based on test values of E in Table 4, the largest difference occurring with the proportional A. C. I. Code values for 2000-lb. concrete where the greatest divergence in E is to be found (see Fig. 4 (b)). Actual percentages of steel, however, vary considerably for the lower strength concretes.

Combined Effect of Lightness in Weight and Low Modulus of Elasticity on Slab and Beam Design. In general a design in Haydite concrete will permit advantage to be taken of reduced dead load and must necessarily be made on the basis of actual values of E. Comparative design ratios are shown in Table 7 based on proportional A. C. I. Code values of E, for all concrete strengths, and for a range of ratios of dead load to total load from 0.50 to 0.20. Percentages of resisting moment required for Haydite to resisting moment required for gravel, $\frac{R.\ M.}{(R.\ M.)_g}$, have been taken from Table 3. Percentages given in section B of Table 7 for slabs with balanced reinforcement show a requirement for "C"-Haydite concrete of 88 to 91 per cent of the depth of gravel concrete slabs with corresponding values of 105 to 108 per cent of the steel areas for gravel concrete. For All-Haydite concrete similar percentages of depth are 80 to 85, with 111 to 117 per cent of steel. The analysis for variable resisting moment is similar to that given for Table 4.

Thus for a variable resisting moment, $R.\ M.$,

Slab comparison:

depth ratios, $$\frac{d}{d_g} = \left(\frac{K_g}{K} \cdot \frac{R.\ M.}{(R.\ M.)_g}\right)^{1/2}$$

and steel ratios, $$\frac{A_s}{A_{sg}} = \frac{d}{d_g} \cdot \frac{p}{p_g}$$

Rectangular Beams Comparison:

concrete area ratios, $$\frac{bd}{(bd)_g} = \left(\frac{K_g}{K} \cdot \frac{R.\ M.}{(R.\ M.)_g}\right)^{2/3}$$

steel ratios, $$\frac{A_s}{A_{sg}} = \frac{bd}{(bd)_g} \cdot \frac{p}{p_g}$$

Rectangular beam comparisons given in section C of Table 7 show area requirements for "C"-Haydite concrete of 85 to 89 per cent with corresponding steel requirements of 101 to 106 per cent, and area requirements for All-Haydite concrete of 74 to 79 per cent, with steel requirements of 103 to 110 per cent, as referred to gravel concrete requirements.

Having obtained the reduced depths shown in Table 7 a further reduction of required resisting moment (due to this decrease in dead load) might be applied thus effecting a further slight reduction of all concrete and steel areas for Haydite concretes.

T-Beams and Beams With Compressive Reinforcement: For a comparison of T-beams of different concretes, since the depth is usually assumed or limited by shear or architectural considerations, only beams of the same stem size will be compared. In general the decrease in bending moment due to reduced dead load for Haydite concretes will be somewhat greater than the decrease in values of j, resulting in a slight saving in steel. A single illustration is given in Table 8 for the design at the center of a T-beam, the problem being taken from "Design and Cost Data for the Building Code," Arthur R. Lord, Proc. A. C. I., Volume 24, 1928, p. 546, problem 2.

In Haydite beams of the same size as gravel the efficiency of compressive steel is increased due to two causes, greater values of n and larger proportions of f_c to be multiplied by these values of n in determining the ability to develop compressive stress in the steel. The larger proportions of f_c result from increased values of k for Haydite concretes and in quantity will depend upon d when the distance from the compression surface to the compression steel is constant.

Shear and Bond: As a result of beam and pull-out tests* to determine bond values and beam tests* to determine diagonal tension resistance of concretes of gravel, "C"-Haydite, and All-Haydite, it was concluded that the same working stresses may be applied to all three concretes for shear and bond.

Deflection: While relative deflections of homogeneous beams of constant size and span will be inversely proportional to their

*Proc. A. S. T. M., Vol. 30, Part II, 1930. "Tests of Plain and Reinforced Haydite Concrete."

E values the same will not be true of non-homogeneous beams of reinforced concrete. This is apparent from a study of Maney's equation† which gives deflection, D, as a function of the separate deformations of the steel fiber, ϵ_s, and the concrete fiber, ϵ_c,

$$D = K\frac{l^2}{d}(\epsilon_s + \epsilon_c)$$

Tests show that beams made with gravel, "C"-Haydite and All-Haydite concrete having relative values of E of 1.00, 0.75 and 0.55, developed relative deflections under a given load of 1.00, 1.15 and 1.30, respectively. This is readily understood by reference to the formula, in which the concrete deformation increases as E decreases, while the steel deformation is only slightly affected by variations in the stiffness of the concrete.

Column Design: In applying a column formula of the type $\frac{P}{A} = f_c[1 + (n - 1)\ p]$ it is obvious that an increase in n for a constant value of f_c and p would result in increasing the working load and that the higher the percentage p the greater would be the increase in working load. A reinforced concrete column made of Haydite concrete of a given strength can not be expected to carry a greater ultimate load than a similar gravel concrete column, identically reinforced. It is apparent that this type of formula embodying n will not apply to a series of concretes having widely differing values of n for identical strengths. Tests of columns of gravel concrete, "C"-Haydite concrete, and All-Haydite concrete covering plain columns, tied columns having 1.5 per cent and 4 per cent longitudinal steel, and spirally reinforced columns having 1 per cent spiral and 1.5 and 4 per cent longitudinal steel have shown that approximately the same relationships may be expected between load carried, ultimate concrete strength and yield point of reinforcement for all three kinds of concrete regardless of the values of E.

In view of the above considerations it seems advisable in the design of axially loaded Haydite columns to make a simple substitution of concretes, taking no account of a reduction in E, and making only such reduction in dead load as the substitution permits. The above equation indicates a distribution in stress

†Proc. A. S. T. M., Vol. XIV, Part II, 1914. "Relation between Deformation and Deflection in Reinforced Concrete Beams." by G. A. Maney.

between the concrete and the steel at working loads which has little real significance. For cases of eccentrically loaded columns in which the bending moment induced results from a rigid connection with a girder or slab, the low modulus of elasticity of the column may be of considerable advantage. As recently noted by Prof. Hardy Cross, when a heavy stiff girder is supported by relatively light columns, the bending moment induced in the column varies almost directly with its stiffness. Such a column may be difficult to design, since any attempt to increase the resisting moment is likely to increase the stiffness and consequent bending moment. A gravel concrete girder supported on Haydite concrete columns might thus represent a logical and sensible design.

SUMMARY

The principal conclusions to be drawn from the foregoing discussion may be summarized as follows:

The relation between compressive strength and water-cement ratio of Haydite concrete does not differ greatly from that for gravel concrete, all test results falling within a narrow zone as shown in Fig. 2. Concretes made with initially moist Haydite were consistently a little stronger than those made with dry Haydite aggregates. Due to their gradation and surface texture, Haydite aggregates in general required a larger water-cement ratio than gravel aggregates to produce a desired workability with a given mix, as indicated by slump and flow tests.

The determination of the water-cement ratio involves an accurate correction for the amount of absorbed moisture. The total absorption of moisture in a given period varies with the initial moisture condition of the Haydite aggregate, and is greater if the material is initially moist than if it is dry.

It is recommended that to secure workability the proportion by volume of coarse Haydite, of the sizes described herein, shall not exceed 55 per cent of the total aggregate. With this proportion, the unit weight of the usual range of structural concretes will be 115 to 120 lb. per cu. ft. for "C"-Haydite concrete and 95 to 100 lb. per cu. ft. for All-Haydite concrete.

The modulus of elasticity of "C"-Haydite concrete is about 75

per cent and that of All-Haydite concrete about 55 per cent, of that for gravel concrete.

With regard to the design of reinforced slabs and beams, there are two features of Haydite concrete that are important, the lightness in weight and the low modulus of elasticity. The first will permit considerable reductions in dead load over ordinary concrete, thus permitting a reduction in size of members or an increase in span, as well as decreasing loads on columns and footings.

The effect of low modulus of elasticity in a reinforced concrete beam or slab is to lower the neutral axis and to require a higher percentage of steel for balanced reinforcement. However, the depth of member may be decreased so that the saving in concrete quantities will offset the increase in steel requirements.

Combining the effects of light weight and low modulus of elasticity and taking advantage of possible reduction in slab depth, it is found that the substitution of "C"-Haydite for gravel concrete will permit a decrease of 12 to 9 per cent of concrete with an increase of 5 to 8 per cent of steel. With All-Haydite concrete there may be 20 to 15 per cent reduction in concrete with 11 to 17 per cent increase in steel. In rectangular beam design the conditions are even more favorable to the use of Haydite concrete.

The working stresses for bond and diagonal tension apparently may be taken as the same percentage of the compressive strength as is usually recommended for ordinary concrete.

The values of modulus of elasticity found for gravel concrete do not agree particularly well with the design values specified by the A. C. I. Joint Building Code, except at strengths of 3500 to 3750 lb. per sq. in. Use of the Code formula, modified by the introduction of 75 and 55 per cent for "C"-Haydite and All-Haydite concretes will produce considerable error for concretes of very high or very low strengths.

Despite the implications of the current column formulas, such as that of the Joint Building Code, the low modulus of elasticity should not be used to justify any increase in working stresses in Haydite columns over those for gravel concrete. Using concrete

of equal compressive strength, equal ultimate column strengths will result. The smaller stiffness of Haydite columns may be of advantage in decreasing column moments in rigid frames and like structures.

Because of the variation of construction costs in different localities, no attempt has been made to carry the foregoing studies through to the ultimate cost of any structure. However enough has been given to indicate the variation in quantities to be expected in certain common structural members. Other less tangible features of the Haydite concrete structure may lie in the possibility of increased building heights, the use of cheaper types of footings, the possibility of greater panel sizes, and the utilization of insulating properties of the material. The saving of weight in a structure such as in a bridge floor or in a longspan roof is of very obvious advantage. Both the properties of the material and its economy in design remain to be explored more fully.

TABLE 1
PROPERTIES OF HAYDITE AGGREGATES

Property	Fine ("A") Haydite	Coarse ("C") Haydite
Unit Weight		
Dry, Rodded, A. S. T. M. Method	60—64	45—49
Dry, Loose measure	50—56	41—45
Moist, Loose, as received	51	46
Moisture Content as received		
Per cent, by weight	2.9—27.0	3.3—7.6
Fineness modulus	2.3—3.0	5.9—6.3
Apparent specific gravity	.....	1.25—1.32

TABLE 2

DATA OF COMPRESSION TESTS OF HAYDITE CONCRETE

Proportions by loose dry volume.
Dry aggregates used.
Water allowed for absorption: Sand, 1 per cent; Fine Haydite, 14 per cent; Coarse Haydite, 7 per cent.
Quantities required per cubic yard of concrete based on fresh concrete.
No allowance has been made for settlement and waste.
All values given are the average of three tests.

Aggregates	Mix	Water Cement Ratio by Vol.	Slump inches	28-Day Cylinders		Quantities Req'd per cu. yd. of Concrete		
				Compressive Strength lb. per sq. in.	Initial Modulus of Elasticity lb. per sq. in.	Cement bbl.	Sand or Fine Haydite cu. yd.	Coarse Haydite cu. yd.
Sand and Coarse Haydite	1:1½:2½	0.77	6.2	4300	2,610,000	1.83	0.41	0.68
		0.87	8.7	3810	2,440,000	1.79	0.40	0.66
		0.97	9.7	2940	2,370,000	1.75	0.39	0.65
	1:2 :2	0.76	3.1	4370	3,010,000	1.81	0.54	0.54
		0.86	8.5	3960	2,780,000	1.78	0.53	0.53
		0.96	10.2	3190	2,600,000	1.73	0.51	0.51
	1:2 :3	0.89	4.6	3640	2,600,000	1.51	0.45	0.67
		0.99	8.3	3110	2,370,000	1.50	0.44	0.67
		1.09	8.6	2530	2,150,000	1.47	0.44	0.65
	1:2½:2½	0.87	2.6	3490	2,680,000	1.50	0.56	0.56
		0.97	7.8	3030	2,530,000	1.48	0.55	0.55
		1.07	8.5	2700	2,310,000	1.43	0.53	0.53
	1:2½:3½	1.10	4.6	2530	2,250,000	1.29	0.48	0.67
		1.20	6.2	2050	2,210,000	1.26	0.46	0.65
		1.30	7.6	1850	2,000,000	1.24	0.46	0.64
	1:3 :3	1.09	3.1	2620	2,450,000	1.27	0.56	0.56
		1.19	5.1	2100	2,220,000	1.25	0.56	0.56
		1.29	6.9	1920	2,250,000	1.24	0.55	0.55
Fine and Coarse Haydite	1:1½:2½	0.86	5.4	3700	2,010,000	1.86	0.41	0.69
		0.96	6.6	3060	1,860,000	1.79	0.40	0.66
		1.06	9.6	2440	1,800,000	1.75	0.39	0.65
	1:2 :2	0.88	5.5	3470	1,950,000	1.83	0.54	0.54
		0.98	7.5	2890	1,670,000	1.77	0.52	0.52
		1.08	9.9	2490	1,770,000	1.73	0.51	0.51
	1:2 :3	1.01	1.9	2530	1,820,000	1.53	0.45	0.68
		1.11	5.3	2300	1,730,000	1.50	0.44	0.67
		1.21	8.4	1920	1,670,000	1.47	0.44	0.65
	1:2½:2½	1.03	1.7	2620	1,850,000	1.51	0.56	0.56
		1.13	5.4	2310	1,810,000	1.50	0.56	0.56
		1.23	7.7	2060	1,630,000	1.46	0.54	0.54
	1:2½:3½	1.25	1.0	1580	1,330,000	1.30	0.48	0.68
		1.35	7.4	1680	1,620,000	1.28	0.48	0.67
		1.45	7.6	1310	1,540,000	1.24	0.46	0.64
	1:3 :3	1.27	2.7	1760	1,690,000	1.29	0.57	0.57
		1.37	5.3	1620	1,310,000	1.28	0.57	0.57
		1.47	6.7	1280	1,570,000	1.23	0.55	0.55

TABLE 3

EFFECT OF LIGHTNESS IN WEIGHT

Percentage of Total Load to be Carried

Kind of Concrete	Relative Unit Weights of Concrete	Dead Load—Percentage of Total Load				
		60	50	40	30	20
Gravel	100	100	100	100	100	100
C-Haydite	80	88	90	92	94	96
All-Haydite	67	80	83	87	90	93

TABLE 4

COMPARATIVE DESIGN OF RECTANGULAR BEAMS AND SLABS BASED UPON EXPERIMENTAL VALUES OF E

Balanced Reinforcement, $f_s = 20{,}000$ lb. per sq. in., $f_c = 0.40\,f'_c$.

Sec.	Property	Gravel Concrete				"C"-Haydite				All-Haydite			
	f'_c	2000	2500	3000	3750	2000	2500	3000	3750	2000	2500	3000	3750
	f_c	800	1000	1200	1500	800	1000	1200	1500	800	1000	1200	1500
A	n	10.1	9.4	8.8	8.0	13.4	12.7	11.9	11.0	18.4	17.1	16.0	14.5
	k	0.288	0.319	0.346	0.375	0.348	0.388	0.417	0.452	0.423	0.461	0.490	0.520
	j	0.904	0.894	0.885	0.875	0.884	0.871	0.861	0.849	0.859	0.846	0.837	0.827
	p	0.0058	0.0080	0.0104	0.0140	0.0070	0.0097	0.0125	0.0169	0.0085	0.0115	0.0147	0.0195
	K	104	143	184	246	123	169	216	288	145	195	245	322
	Slab Comparison Based upon Constant Resisting Moment. b constant, d varies												
B	$\frac{d}{d_g}$ percent	100	100	100	100	92	92	92	91	84.5	86	87	87.5
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	111	111	110.5	110	123.5	124.0	122.5	121.5
	Slab and Rect. Beam Comparison Based upon Constant b and d for All Concretes.												
C	$\frac{K}{K_g}$ percent	100	100	100	100	118	118	117	117	139	136	133	131
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	121	121	120	121	146	144	141	139
	Rectangular Beam Comparison Based upon Constant Resisting Moment and upon Proportional b and d for Haydites and Gravel												
D	$\frac{bd}{(bd)_g}$ percent	100	100	100	100	89.0	89.5	90.0	90.0	80.0	80.5	82.5	83.5
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	107	108	108	109	117	116	117	116

Note: Subscript g denotes gravel concrete.

TABLE 5

COMPARATIVE DESIGN OF RECTANGULAR BEAMS AND SLABS BASED UPON VALUE OF E FROM A. C. I. CODE

$f_s = 20{,}000$ lb. per sq. in., $f_c = 0.40\,f'_c$.
Balanced Reinforcement

Sec.	Property	Gravel Concrete $E_g = 1000\,f'_c$				"C"-Haydite $E_h = 750\,f'_c$				All-Haydite $E_a = 550\,f'_c$			
A	f'_c	2000	2500	3000	3750	2000	2500	3000	3750	2000	2500	3000	3750
	f_c	800	1000	1200	1500	800	1000	1200	1500	800	1000	1200	1500
	n	15	12	10	8	20	16	13.3	10.7	27	21.8	18.2	14.5
	k	0.375	0.375	0.375	0.375	0.445	0.445	0.445	0.445	0.521	0.521	0.521	0.521
	j	0.875	0.875	0.875	0.875	0.852	0.852	0.852	0.852	0.826	0.826	0.826	0.826
	p	0.0075	0.0094	0.0112	0.0140	0.0089	0.0111	0.0133	0.0167	0.0104	0.0130	0.0156	0.0195
	K	131	164	197	246	151	189	227	284	172	215	258	323
	Slab Comparison Based upon Constant Resisting Moment. b constant, d varies.												
B	$\frac{d}{d_g}$ percent	100	100	100	100	93	93	93	93	87.5	87.5	87.5	87.5
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	111	111	111	111	121.5	121.5	121.5	121.5
	Slab and Rect. Beam Comparison Based upon Constant b and d for all Concretes												
C	$\frac{K}{K_g}$ percent	100	100	100	100	115	115	115	115	131	131	131	131
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	119	119	119	119	139	139	139	139
	Rectangular Beam Comparison Based upon Constant Resisting Moment and upon Proportional b and d for Haydite and Gravel Concretes												
D	$\frac{bd}{(bd)_g}$ percent	100	100	100	100	91	91	91	91	83	83	83	83
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	108	108	108	108	115	115	115	115

Note: Subscript g denotes gravel concrete.

TABLE 6

COMPARATIVE DESIGN OF RECTANGULAR BEAMS AND SLABS BASED UPON CONSTANT VALUE OF E

Balanced Reinforcement, $f_s = 20{,}000$ lb. per sq. in., $f_c = 0.40\,f'_c$.

Sec.	Property	Gravel Concrete $E_g = 3{,}400{,}000$ $n_g = 8.8$				"C"-Haydite $E_h = 2{,}500{,}000$ $n_h = 12.0$				All Haydite $E_a = 1{,}900{,}000$ $n_a = 15.8$			
	f'_c	2000	2500	3000	3750	2000	2500	3000	3750	2000	2500	3000	3750
	f_c	800	1000	1200	1500	800	1000	1200	1500	800	1000	1200	1500
A	k	0.260	0.306	0.345	0.398	0.325	0.375	0.419	0.474	0.388	0.442	0.487	0.543
	j	0.913	0.898	0.885	0.867	0.892	0.875	0.860	0.842	0.871	0.853	0.838	0.819
	p	0.0052	0.0076	0.0103	0.0149	0.0065	0.0094	0.0126	0.0178	0.0078	0.0110	0.0146	0.0204
	K	95	137	183	258	116	164	216	299	135	188	245	333
	Slab Comparison Based upon Constant Resisting Moment. b constant, d varies												
B	$\frac{bd}{(bd)_g}$ percent	100	100	100	100	90.5	91.5	92.0	93.0	84.0	85.5	86.5	88.0
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	113.0	113.5	112.0	111.5	126.0	124.0	123.0	120.5
	Slab and Rect. Beam Comparison Based upon Constant b and d for all concretes.												
C	$\frac{K}{K_g}$ percent	100	100	100	100	122	120	118	116	142	137	134	129
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	125	124	122	120	150	145	152	137
	Rectangular Beam Comparison Based upon Constant Resisting Moment and upon b and d for Haydites proportional to b and d for gravel.												
D	$\frac{bd}{(bd)_g}$ percent	100	100	100	100	87.5	89.0	89.5	90.5	79.0	81.0	82.0	84.5
	$\frac{A_s}{A_{sg}}$ percent	100	100	100	100	109	110	109	108	118	117	116	115

Note: Subscript g denotes gravel concrete.

TABLE 7

EFFECT OF LIGHTNESS IN WEIGHT AND LOW VALUE OF E

Comparative design of Slabs and Rectangular Beams Based upon Proportional A. C. I. Code Values of E.

Balanced Reinforcement, $f_s = 20{,}000$ lb. per sq. in., $f_c = 0.40\, f'_c$.

All values of f'_c included.

For values of n, k, j, p, and K see Table 5.

Sec.	Property	Gravel Concrete $E_g = 1000\, f'_c$				"C"-Haydite Concrete $E_h = 750\, f'_c$				All-Haydite Concrete $E_a = 550\, f'_c$			
A	$\frac{\text{Dead Load}}{\text{Total Load}}$, per cent	50	40	30	20	50	40	30	20	50	40	30	20
	$\frac{R.\,M.}{(R.\,M.)_g}$, per cent (See Table 3)	100	100	100	100	90	92	94	96	83	87	90	93
	Slab Comparison Based upon Reduced Resisting Moment Req'd. b constant, d varies.												
B	$\frac{d}{d_g}$, per cent	100	100	100	100	88	89	90	91	80	81	83	85
	$\frac{A_s}{A_{sg}}$, per cent	100	100	100	100	105	106	107	108	111	113	115	117
	Rectangular Beam Comparison Based upon Reduced Resisting Moment. Proportionally Reduced b and d.												
C	$\frac{bd}{(bd)_g}$, per cent	100	100	100	100	85	86	87	89	74	76	78	79
	$\frac{A_s}{A_{sg}}$, per cent	100	100	100	100	101	102	104	106	103	106	108	110

TABLE 8

COMPARATIVE DESIGN AT CENTER OF T-BEAM

Slab, 4-in. total thickness.
Stem, 12 x 16 inches.
Beam spacing, 6 ft. centers.
f'_c = 3000 lb. per sq. in.
f_s = 20,000 lb. per sq. in.

Property	Kind of Concrete		
	Gravel	C-Haydite	All-Haydite
Total load—lb. per sq. ft.	300	290	285
Slab load—per lin. ft. of bm.	1800	1740	1710
Weight of stem	150	120	100
Total load, W—lin. ft. of bm.	1950	1860	1810
W/W_g, per cent	100	95	93
B. M., $\frac{Wl^2}{16}$ in. lb.	845,000	806,000	784,000
j	0.884	0.875	0.870
j/j_g, per cent	100	99	98.5
A_s, sq. in.	3.52	3.41	3.34
A_s/A_{sg}, per cent	100	97	95

Readers are referred to the JOURNAL *for February, 1931, for discussion which may develop. Such discussion should reach the Secretary by January 1, 1931.*

Blast Furnace Slag as Concrete Aggregate

Report of Committee 201, Aggregate Specifications

Former Institute Committee E-5, Aggregates, assigned its subcommittee on Slag, H. J. Love, Chairman, to report a summary of available information on Blast Furnace Slag as Concrete Aggregate. The report here presented, as the result of that assignment, has been approved by the successor Committee 201, Aggregate Specifications, H. F. Clemmer, Chairman, and is here offered for the information of Institute members and others and discussion invited.—Editor

1. Definition of blast furnace slag

"Blast furnace slag is the non-metallic product, consisting essentially of silicates and alumino-silicates of lime, which is developed simultaneously with iron in the blast furnace." (Proc., Amer. Con. Inst., page 524, 1925).

2. Introduction

The technical committees of practically all the national organizations interested in construction materials have approved the use of slag aggregate in specifications or reports, including the following:

Amer. Concrete Institute; Amer. Soc. for Testing Materials; Amer. Railway Engineering Assn.; Amer. Soc. for Municipal Improvements; Associated Tile Manufacturers; National Paving Brick Manuf. Assn.; Portland Cement Assn.; Asphalt Institute; Amer. Assn. of State Highway Officials; Federal Specifications Board; U. S. Bureau of Roads (Dept. of Agriculture); U. S. Bureau of Standards (Dept. of Commerce); State Highway Departments of Alabama, Delaware, Dist. of Columbia, Florida, Georgia, Kentucky, Maryland, Mississippi, Missouri, New York, New Hamsphire, South Carolina, Ohio, Pennsylvania, Utah and West Virginia.

Although the tonnage of slag used is only about 4 per cent of the estimated total of all aggregates, this amount is confined almost exclusively to the large centers of population having steel mills, so that the use of slag becomes more important than is

indicated by its comparative tonnage. With this in mind, the Committee has made first-hand investigations of the properties and uses of blast furnace slag as revealed in (a) Actual service in the field and (b) Test data and technical literature. The latter have been made available to the Committee by the National Slag Association through its symposiums, which have been issued regularly during the last two years for the benefit of national technical societies engaged in formulating specifications.

On one inspection trip, 115 miles of slag concrete pavement were inspected, in addition to slag concrete factory buildings, office buildings, residences, bridges, water tanks, boiler stacks and fences. The latter item, fences, included the mile-long cast-in-place fence along the property of the Carnegie Steel Co., Youngstown, O., which has been standing for 13 years. Eight bridges were inspected on this trip, including the 16-year old North Howard bridge at Akron, O., the 15-year Lake Milton bridge west of Youngstown, O., the 14-year West Summit St. bridge at Warren, O., the North Hill Viaduct in Akron, and Lanternman's Falls bridge in Youngstown.

The interest shown in slag aggregate is evinced by papers published in the Proceedings of the American Concrete Institute:

"Slag as Concrete Aggregate," by Sanford E. Thompson, Vol. 13, p. 107.
"Tests of Slag as the Coarse Aggregate in Concrete," by P. J. Freeman, Vol. 14, p. 95.
"Fire Tests of Concrete Columns," by Walter A. Hull, Vol. 14, p. 138.
"Fire Tests of Concrete Columns," by Walter A. Hull, Vol. 16, p. 20.
"Flexural Strength of Plain Concrete," by Duff A. Abrams, Vol. 18, p. 20.
"Report of Fire Resistance of Concrete Building Units," by A. C. I. Committee P-5, Vol. 19, p. 331.
"Proportioning Concrete Materials," by George W. Hutchinson, Vol. 21, p. 148.
"Concrete Pavement Design," by L. W. Teller and J. T. Pauls, Vol. 22, p. 314.
"Tests of Retempered Concrete," by H. F. Gonnerman and P. M. Woodworth, Vol. 25, p. 344.

In addition to these papers, the Institute recognized slag aggregate in the following specifications:

"Proposed Specifications for Sidewalks and Floors," 1919.
"Standard Building Regulations for Reinforced Concrete," 1920.
"Standard Specifications for Monolithic Concrete Sewers and Reinforced Concrete Pipe Sewers," 1920.

"Standard Specification for Cement Concrete Pavements," 1924, revised in 1925.

"Tentative Purchase Specifications for Concrete Aggregates," 1929.

3. Processing blast furnace slag

The processing of slag really begins in the blast furnace, where the iron and slag are produced simultaneously. Layers of coke, flux stone and ore are placed in the furnace through an opening in the top, and the mass subjected to a blast of hot air. During combustion of the coke, the gangue or dross of the ore separates from the metallic iron and combines with the flux stone, and it is this combination, along with certain non-combustible material in the coke, which forms blast furnace slag. At the base of the furnace the molten slag and iron automatically separate due to difference in specific gravity, the slag floating on the iron. The slag is drawn off between iron flushes through the "slag notch" in the side of the furnace above the level of the molten iron. The iron is drawn off through a lower hole (the "iron notch") on the other side of the furnace, followed by any slag which has accumulated since the last slag flush.

Methods of Cooling

Slags are of two classes with regard to method of handling on leaving the furnace:

1—Slag which is brought from molten to solid form entirely through the agency of atmospheric air, known as air-cooled blast furnace slag.

2—Slag which is changed from molten to solid form by means of water or other rapid-cooling agency, known as granulated slag.

Although by far the greater portion of commercialized slag is in the air-cooled form, granulated slag finds an extensive use for the following purposes:

Fills, including backfilling.
Subgrade soil corrective in pavement construction.
Sub-base (blanket) course under all types of pavement.
Bedding course under brick pavement.
Granulated slag roads.
Aggregate in concrete products of all kinds.
Soil corrective in agriculture.
Raw material in manufacture of portland cement.

In Europe slag has been granulated by water, steam, and air under pressure, but American practice invariably makes use of

water. The molten slag is conducted in runners to a pit containing a small amount of water, where it may be struck by a stream of water under pressure, or may be granulated merely by falling into the pit. It can then be removed immediately with grab-buckets. Except for chemical composition, granulated slag is an entirely different product from air-cooled slag, being highly porous, easily crumbled in the hand, and occurring in sharp grains. It is gray or yellow in color. Except for concrete products, it is little used as concrete aggregate.

Methods of Disposal

Commercial slag aggregate for concrete is produced from air-cooled slag. From the furnace the molten slag may follow one of two courses: It may flow through runners direct to small pits alongside the furnace (known as the pit system of disposal), or it may be poured into large ladles mounted on standard railway trucks and conveyed to a more distant point for disposal. The latter operation, in turn, may take one of two methods:

1—The slag may be poured from the ladles into large, well-defined pits served by permanently-located pouring and loading tracks, from which it is regularly excavated (known as the modified pit system).

2—The slag may be poured from the ladles over a large plot of land with no very definite attempt to control width or depth of the solid slag so formed (known as the bank system).

Modern practice follows for the most part either the pit or modified pit system, each of which has its peculiar advantages. The pit system calls for a small percentage of the area demanded by the modified pit system, but that area must be alongside the furnace, which is not always feasible. The modified pit system, covering a larger area, has the advantage of providing greater storage facilities for the slag, so that the crushing and screening plant need not be running continuously as with the pit system. The bank system calls for large areas of land and has other disadvantages making for its disuse.

The molten slag is poured into the pits in a thin layer, which brings about a rapid drop in temperature and the solidification of the material. Succeeding layers retain some heat in the mass and a tempering action takes place, resulting in a hard, tough

product suitable for processing into concrete aggregate. When one pit has been filled by successive pourings, the filling of another begins, while a power shovel may enter the full pit and excavate the solidified slag. Where the pit system is in use, the shovel follows the filling of the pit very closely, but in large modified-pit layouts six months may elapse between filling and excavating. The time the slag lies in pits before use depends on the size of the pits and the demand for the product. At one time it was thought advisable to specify that the slag lie in the pits for a definite period of "aging," but tests and observations[15]* have shown this to be unnecessary, and now there are excavated and used annually in Ohio alone 1,250,000 tons of slag less than two weeks old. The total production for commercial purposes throughout the United States approximates 10,000,000 tons annually, about thirty per cent of which is used as concrete aggregate.

Crushing and Screening Operations

The process of cooling in layers provides planes in the slag mass which assist materially in excavation, and blasting is very seldom resorted to. The slag as excavated is loaded into dump cars and carried to the crushing and screening plant, where it passes through the processes typical of a plant preparing crushed aggregate for the market. There is one exception, however,—the slag is passed over magnetic separators during the operation in order to remove any small pieces of metallic iron carried over into the slag at the furnace. These are returned to the steel companies and reclaimed. After the screening operation has been completed, the slag is chuted to bins from which railroad cars or motor trucks may be filled.

4. Physical and chemical characteristics of slag

Air-cooled blast furnace slag as produced for commercial purposes is a rough, angular aggregate varying in color from light to dark gray, depending upon the character of the materials charged to the furnace. The roughness is due to its cellular structure, and the degree of roughness varies as the cells are large or small, many or few. These cells are formed by gases occluded in the slag at time of cooling, and are generally non-communicating; hence the term "cellular" or "vesicular" is more applicable than the term "porous." As excavated from the pit or bank,

*All references are to the bibliography at end of report.

slag breaks into roughly cubical pieces containing practically no flat pieces or slivers. This angularity, combined with its rough surface texture, gives slag a larger surface area per unit of volume than is usually found in concrete aggregates.

When fresh, there is a perceptible odor from blast furnace slag due to the small amount of sulphur compounds which it contains (1.00-1.75 per cent), but later this almost entirely disappears. Since it has passed through the high temperatures of the blast furnace, slag is a particularly clean material, and it is dug from pits in which nothing but slag is placed. It is also clean from the standpoint of adherent dust-coating, since this is readily removed during the screening operation.

Specific Gravity

The apparent specific gravity of air-cooled blast furnace slag as determined by the standard method averages about 2.25, although samples will be found ranging from 1.80 to 2.80 due to the cellular structure of the material. The true specific gravity of the solid material is fairly constant between 3.00 and 3.10.

Weight per Cubic Foot

The difference in specific gravity of various pieces of slag inevitably affects the weight of the aggregate per cubic foot,—if the unit of volume contains a large percentage of either light or heavy pieces, the weight may vary from 65 to 100 lb. per cu. ft. in the concrete aggregate sizes. The higher weights are relatively rare and represent only a very small per cent of the total tonnage. In general, the highest weight in the concrete aggregate sizes is about 85 lb. The foregoing weight values are based on the aggregate compacted in a dry condition; if the material is weighed loose, the values will be approximately 10 lb. per cu. ft. lower.

Voids and Absorption

Void determinations by the water displacement method show a void content of 40-45 per cent (average 42 per cent) in slag sized for concrete aggregate. If the determination is made by calculation from the specific gravity and weight per cubic foot, the result will be approximately the same if the specific gravity has been based on a representative sample. The presence of light or heavy pieces in practically all blast furnace slags makes

most specific gravity determinations of doubtful value and also greatly affects the absorption determination, since a few pieces of such slag will affect the absorption appreciably. Furthermore, the small sample chosen for absorption and specific gravity determinations may contain a much larger percentage of light or heavy pieces than the stockpile, bin or car of which it is supposed to be representative. Absorption tests show an average of about 4 per cent by weight with a high and low range of 25.0 to 0.25 per cent, with the great majority lying within the field of 2-4½ per cent absorption. Although these average figures appear somewhat high in comparison with other aggregates, the actual amount of water absorbed may be no greater than that indicated by a lower percentage on other aggregates, due to the latter having specific gravities averaging 20 per cent higher than slag. For example, a slag of specific gravity 2.21 and an absorption by weight of 6 per cent has the same absorption *by volume* as an aggregate of specific gravity 2.65 and an absorption by weight of 5 per cent.

The absorption which takes place in the first 30 minutes is usually that considered in proportioning slag concrete, and allowance must be made for this amount if the mixing water is to be kept under control.

Hardness and Toughness

Hardness and toughness factors have been found meaningless when applied to slag aggregate, and are no longer required in specifications.

Resistance to Abrasion

Testing slag for resistance to abrasion in the Deval machine is subject to the same difficulties as the specific gravity and absorption determinations. The small sample may contain many more of the light or heavy pieces than the pile from which it was taken, and these pieces will materially affect the test results. It is further believed that neither stone nor slag is adapted to such a test, since the sharp edges are rapidly broken off and appear as "loss by abrasion," even though such breakage represents a large part of that which occurs throughout the duration of the test. That this actually happens has been proven by tests on slag,[1] in which the sharp edges were first worn off in the machine, and the

material then subjected to the standard Deval test. The results showed that a slag which averaged 13.2 per cent loss in the natural state averaged only 8.2 per cent loss with the sharp edges removed. The standard Deval test, which is used only on large-sized pieces of aggregate, gives test results on slag varying from 2½—43 per cent, but the average loss lies within the limit of 12—16 per cent.

The modified Deval abrasion test, in which steel balls are introduced as an abrasive charge, is used in Ohio for testing the concrete aggregate sizes. The wear is proportionately greater, averaging about 25 per cent loss with a range of 15-38 per cent.

The abrasion factor as a specification requirement on slag is being eliminated from most specifications, following the path of hardness and toughness factors. The inaccuracy of the Deval abrasion test as applied to slag is strikingly brought out in data recently presented to the American Association of State Highway Officials. These data were obtained by 17 well known laboratories coöperating in tests of slags from four different sources. In order to insure uniformity of the samples, they were selected from each source by A. S. Rea, Chief Engineer of Tests, Ohio Highway Dept., and shipped to the respective laboratories for testing. The abrasion tests were run according to the Standard Deval method (A. S. T. M. Serial Designation D 2-26). When the results were reported, a decided variation among laboratories was evident. In the following table is summarized (a) The maximum, minimum and average of all laboratories for each slag, (b) The maximum variation from the average shown by any one laboratory, and (c) The mean variation from the average. The latter was arrived at by totalling the individual variations from the average, and dividing the sum by seventeen.

Slag	Abrasion Loss			Variations from Average Per Cent	
	Max.	Min.	Ave.	Maximum	Mean
A	23.0	9.1	13.6	69	15
B	21.5	7.8	13.2	62	14
C	18.3	5.1	12.9	60	15
D	6.6	1.0	4.7	78	18

These large variations from the average were not confined to any one laboratory; in general variations of 10 per cent or more

were obtained with all four slags in most of the laboratories.

To determine whether abrasion test results could be duplicated in the same laboratory, seven of the 17 laboratories were asked to run check samples. These consisted of samples designated Slags E, F, G and H, which were selected by Mr. Rea at the same time and from the same source as Slags A, B, C and D, respectively, although this fact was not known to the cooperating laboratories.

Considered as a separate series of tests, the results obtained on Slags E, F, G and H by the seven laboratories show equally as much variation between laboratories as did Slags A, B, C, and D. Tabulated in the same manner as the results from the latter, Slags E, F, G and H showed the following variations:

Slag	Abrasion Loss			Variations from Average Per Cent	
	Max.	Min.	Ave.	Maximum	Mean
E	14.5	3.9	11.7	67	22
F	15.1	9.8	12.6	22	12
G	13.7	4.1	11.4	64	21
H	6.0	0.4	4.0	90	30

In view of the foregoing, it is evident that the Standard Deval abrasion test will not develop concordant results on uniform samples of blast furnace slag, and therefore should be omitted from specifications for slag aggregate.

Crushing Strength

The crushing resistance of slag is about that of the more common aggregates,—approximately 9,000-12,000 lb. per sq. in. —but this factor is of minor importance since it is well above the strength required in concrete.

Cementation Value

The cementation value of blast furnace slag, as measured by the cementation test (described in U. S. Dept. of Agriculture Bulletin 347) varies according to the chemical composition, slags containing a large percentage of calcium oxide having a greater cementing value. Thus there is record of cementing values on blast furnace slag from 3 to 600, indicating the number of blows withstood by a cylinder made of a mixture of finely ground slag and water, the weight of the hammer being 1 kgm. and the height of fall 1 cm.

Melting Point

With regard to the melting point of slag, tests have shown it to be fluid between 2300-2600° F., viscous at 2200° F., and solid at 2100° F. or lower.

Chemical Composition

Chemically, blast furnace slag is made up of calcium oxide (CaO), alumina (Al_2O_3), silica (SiO_2), and magnesia (MgO); there are also small quantities of iron oxide (FeO), manganese oxide (MnO), and sulphur compounds. The chemical composition is entirely dependent upon the nature of the materials charged to the furnace,—the lime and magnesia in the flux stone, the alumina and silica in coke and ore. Although this brings about a difference in chemical analysis for slags from different sources, the slag from any one source maintains a marked degree of uniformity. Thus, monthly analyses of slags from 24 sources in Ohio, Pennsylvania and Kentucky over a 12-month period in 1927 showed mean variations from the average as indicated in the following tabulation:

CONSTITUENT	AVERAGE OF ALL TESTS	MEAN VARIATION FROM AVERAGE
	Per Cent	Per Cent
SiO_2	36.44	2.38
Al_2O_3	12.73	2.13
CaO	41.28	1.83
MgO	6.50	1.20

The relatively high variation in magnesia content (1.20 in 6.50) is due to the use of magnesian limestone or dolomite as flux at some furnaces, while others use nothing but limestone. The general range in chemical composition of blast furnace slag is within the following limits:

SiO_2	32–40 Per Cent
Al_2O_3	10–17 "
CaO	30–49 "
MgO	3–14 "
FeO	.5–2 "
Sulphur	1–2 "

Petrographic Composition

Petrographically, slag is composed mainly of silicates, such as di-calcium silicate and calcium alumino-silicate. There have also been found melilite, periclase, spinel, forsterite, pyroxene,

cordierite, mullite, christobilite, tridymite, anorthite, gehlenite, olivine and akermanite.

5. Properties of slag concrete

Weight

The weight of slag concrete varies from 127 to 150 lb. per cu. ft. according to the mix used and the weight of the slag aggregate itself. Rich mixes containing more of the relatively heavy cement and sand naturally weigh more than the leaner mixes. The figure of 135 lb. per cu. ft. may be taken as average for the intermediate mixes (1:2:3, 1:2½:4 etc.) when a medium-weight slag (70-80 lb. per cu. ft. compact) is used as coarse aggregate.

Absorption

The absorption of any concrete is more affected by the mortar, the cement content, the water content, and the density of the concrete than by the type of aggregate used. The average of all available absorption tests on slag concrete show a value of 5.5 per cent by weight. As in the case of absorption determinations on slag aggregate, this value is not as high as it may appear, since any stated weight of water absorbed will show up as a larger percentage of absorption in slag concrete than in other concretes weighing 10-15 lb. per cu. ft. more. Five and one-half per cent absorption in slag concrete corresponds to 5 per cent absorption in the heavier concretes.

Bond

When "bond" is spoken of in connection with concrete, it may mean either of two properties: (a) Adhesion between concrete and reinforcing steel, or (b) Adhesion between mortar and aggregate. Bond of steel with concrete, being almost entirely a function of the mortar, is the same for slag concrete as other concretes.

With respect to adhesion of mortar and aggregate blast furnace slag possesses several advantages. The rough pitted surfaces and the angularity of the fragments each aids the mortar in obtaining a high degree of bond, so that tests on slag concrete very seldom show failure of the bond between mortar and slag. Such a bond is never the weak point in a slag concrete. Quoting Sanford E. Thompson[3] on this subject:

Inspection of the crushed specimens (of slag concrete) where breaks occur through the pieces shows a very strong bond between the mortar and the pieces of slag. While this probably is due in part to the rough nature of the surface of the slag, it is evident from inspection that a chemical action has taken place between the slag and the mortar. This is shown by a circle of a different color around the edges of the sections of slag.

Durability

The durability of slag concrete has been tested in thousands of structures over a period of 50 years, including buildings, machinery foundations, bridges and pavements. The durability of the aggregate itself has been proven by its successful use as ballast, road material and sewage filtering medium, and also by various test methods. The most prominent of the latter are the sodium sulphate test and the freezing and thawing test. Raymond Harsch[4] has reported sodium sulphate tests on slags from 32 sources in the United States, all the samples being unaffected after 5 cycles of the test.

C. C. Hommon[5] selected thirty-seven samples of slag which had been used as sewage filtering media in as many beds throughout the eastern and southern states. Ten pieces from each of these samples were subjected to 20 cycles of the sodium sulphate test. Although the material had been exposed to the effects of sewage and weather for periods varying from 1 to 21 years, this admittedly severe test had little effect on them, as shown in the accompanying table. The four samples which showed any effect whatever were aged respectively 6, 8, 12 and 14 years. The following table, which includes stones and gravel sampled and tested during the same investigation, shows the results of the tests:

AGGREGATE	NUMBER OF PIECES				PIECES UNAFFECTED
	Tested	Checked	Chipped	Split	Crumbled	Per Cent
Slag	340	0	0	0	0	100
Limestone	20	0	0	0	0	100
Granite	30	0	0	0	0	100
Traprock	10	0	0	0	0	100
Slag	30	1	1	1	0	90
Granite	10	0	0	1	0	90
Slag	10	0	0	2	0	80
Feldspar	10	2	0	0	0	80
Gravel	10	0	0	2	0	80

Limestone	10	2	0	2	0	60
Limestone	10	3	0	0	2	50

In tests by George B. Gascoigne[6], six slags from as many sources were subjected to 10 cycles of sodium sulphate, with result that three pieces of slag out of 36 split, but none of the others were affected in any way. In addition, three of the slag samples were carried to 20 cycles; of the 18 pieces tested at 20 cycles, 6 were unaffected, 4 showed checking, 4 splitting and 4 cracking, but none of the pieces crumbled.

The most extensive series of sodium sulphate tests on record has been carried out by Fred Hubbard[7] on slags from 44 sources in Ohio, Pennsylvania, Maryland and Kentucky. Each of the 422 samples tested underwent 5 cycles of sodium sulphate without showing any signs of checking or cracking, representing a total of 4220 separate pieces of slag. In addition, 358 of the samples (3580 pieces) were carried through to 20 cycles. Of these, 3550 remained intact, denoting a remarkable resistance to the severe action of expanding sodium sulphate crystals. The 30 pieces which did not sustain 20 cycles without some breakdown showed a clean splitting off of one or more fragments in practically every case, there being only three pieces in which there was any crumbling of the material. In terms of percentage, this amounts to 0.08 per cent crumbling material—a negligible quantity.

Freezing and thawing tests of slag have been reported by Edward Orton Jr.,[8] showing a loss of only 0.65 per cent after 30 alternations. G. B. Gascoigne's[6] report also includes the results of such tests, carried to 100 cycles on 2-3 in. pieces as shown in the following table:

AGGREGATE	PIECES TESTED		CONDITION FOLLOWING TEST			
	No.	Weight, Gm.	Pieces over 1″ No.	Pieces over 1″ Weight, Gm.	Fines under 1″ Weight, Gm.	Fines under 1″ Per Cent by Wt.
Slag—1	27	3250	28	3168	82	2.5
Slag—2	22	2993	25	2864	129	4.3
Slag—3	39	3320	40	3209	111	3.3
Slag—4	20	3780	29	3594	186	4.9
Slag—5	18	3966	27	3877	89	2.2
Slag—6	25	3147	29	2935	212	6.7

Tests at the U. S. Bureau of Standards[9] included not only freezing and thawing, but boiling and drying, sodium chloride, and sodium sulphate. Each aggregate was from a different source—6 slags, 3 granites, 4 traprocks, 4 sandstones, 7 limestones and 12 gravels. At the time of the last report, 24 cycles of freezing and thawing had been completed on 28 of the 36 aggregates, and 40 cycles of the other three methods on all 36 aggregates. The report states in part:

> Of the types of aggregates used in these tests, the limestones are apparently showing the greatest disintegration. With few exceptions, by far the greater number of pieces of gravel are sound. Granites have not been affected, and only occasional pieces of trap, sandstone and slag have shown signs of breaking up under these tests.

Probably the best test of the durability of any aggregate is its behavior when used as a filtering medium in sewage beds. Here the surface pieces receive every possible temperature extreme, in addition to alternate freezing and thawing, alternate wetting and drying, and the destructive effects of the sewage itself, which is often increased by the presence of industrial wastes such as acid pickle-liquor from steel mills, tannery wastes, etc. Yet the recent survey of C. C. Hommon[5] showed very little disintegration of the slag filtering medium in 37 sewage beds, 16 of which had been in service for 10-21 years. This survey covered all the slag filter beds which could be found in the United States, and particular attention was paid to the three possible forms of disintegration—splitting, crumbling and flaking. At 29 of the plants, crumbling and flaking were either not present or it was necessary to examine very closely to find them. At 26 plants the same condition obtained with regard to splitting. At the plants where splitting and crumbling could be found, the total amount was comparatively small and would have no effect on the operation of the filters. Although more pieces of slag were affected by flaking, the total amount of such flaking did not represent any greater disintegration than by splitting and crumbling.

The durability of slag under sewage bed conditions has been tested by George. H. Gascoigne.[6] Samples of the material were placed in colanders on the surface of an operating sewage bed within range of the spray from the distributing nozzles. The

samples were left on the bed from Jan. 1927 till Feb. 1928, in which time there were 37 days during which the temperature at all times was below freezing point. The minimum atmospheric temperature was —8° F. and the maximum 99° F. Alternate wettings and dryings occurred on the bed 40-50 times daily during the period. The original size of the aggregates was 2-3 in., and the following table shows the results of the test. The fine material below $\frac{3}{16}$ in. (lost through the holes in the colander) was not calculated:

AGGREGATE	PIECES TESTED		CONDITION AFTER TEST		
	No.	Weight, Gm.	Pieces over 1″	Fines $\frac{3}{16}$″ –1″ Weight, Gm.	Per Cent by Wt.
Slag—1	20	3117	20	0	0
Slag—2	20	2939	20	22	.7
Slag—3	20	1938	20	10	.5
Slag—4	20	3969	20	5	.1
Slag—5	20	3336	20	18	.5
Slag—6	20	2970	20	5	.1

In view of the behavior of slag in the accelerated durability tests just outlined, it would seem superfluous to state that it is a durable and stable material, but there is one small point in this connection which may need clarification. L. G. Carmick's[10] investigations show that slag which is solid at atmospheric temperatures (in other words, as delivered on the job) will remain entirely stable and durable, but occasionally there is produced a small quantity of "limy" slag which crumbles to powder during cooling if it is not processed correctly. It has been found that slow cooling will cause the calcium orthosilicate in certain slags to pass through three forms (alpha, beta and gamma), the last of which involves a volume increase of about 10 per cent and disintegration of the slag. However, if the general practice is followed and the molten slag is poured in thin layers so as to chill quickly and solidify it, the calcium orthosilicate is arrested at the beta form, which is entirely stable and involves no volume change. Tests have proved that any disintegration which takes place does so before the slag has cooled, hence no slag likely to crumble will be found in the finished commercial product. The small amount of fine material developed in the pits by crumbling is removed by the screening opera-

tions, and furthermore has reached its final form and is entirely inert as proved by tests. C. C. Hommon[5] reports a case where hot slag which was visibly crumbling was immersed in water and the crumbling was completely arrested. Further, the cooled slag withstood 5 cycles of sodium sulphate successfully. He also cites the instance of a large piece of slag, part of which had crumbled during cooling; the solid material remaining after cooling withstood 20 cycles of sodium sulphate. The final proof of the relative unimportance of crumbling slag is the fact that there has never been any crumbled or disintegrated slag found in slag concrete.

Like slag aggregate, slag concrete receives its most severe durability test in connection with sewage disposal beds. Concrete is used for the walls and floors of the beds, and also for primary and secondary settling tanks. All these receive the weathering that any exposed concrete is subject to, and in addition the wetting and drying, freezing and thawing caused by the conditions under which such plants are operated. The character of the sewage itself probably has a decided effect in bringing about disintegration, but slag concrete has been used with entire satisfaction for such construction in 19 sewage beds in the United States, including the temperature ranges to be found in New York, Pennsylvania, Ohio, Alabama and Tennessee.

A report of the A. S. T. M. Committee on Concrete Aggregates[11] makes the following conclusive statement with regard to the durability of slag concrete:

> A large amount of test data has shown this material to be acceptable as concrete aggregate, and there is also a large amount of data from the field in testimony of the permanence and durability of slag concrete structures.
>
> No restriction has been placed upon the sulphuric content of slag, for the reason that inspections made by members of the Committee of reinforced slag concrete structures in the course of demolition showed no corrosion of reinforcement that could be attributed to the slag, nor is there any published evidence that such corrosion has been observed so far as the Committee is aware.

The last sentence of the foregoing statement by Committee C-9 answers a question which has often been raised by engineers: "Does slag concrete have any corrosive effect on steel reinforcement?" There has never been a case where slag has been shown

to have affected the reinforcing steel in any way, and many concrete authorities have expressed a decided belief that such action cannot happen. If the concrete is dense and the steel placed at a proper depth from the surface, every possible protection has been afforded, and the steel will be unaffected.

If slag has any corrosive effect on steel, it would have come to light in connection with other uses of the material than reinforced concrete, where the steel is protected from air and moisture. For example, in sewage disposal beds the feeder and riser pipes are covered with slag to a depth of 5-8 ft., and liquid sewage is applied continuously, giving rise to the very conditions which should produce decided deterioration—air, moisture, and even acids. The liquid running down over the slag and thence over the piping undoubtedly produces a more severe condition tending to affect the iron than exists in reinforced concrete, and yet there is a total of 37 slag sewage disposal beds in the United States with service histories up to 21 years, in none of which are there any recorded data that the pipes deteriorated more quickly than usual. This is in spite of the fact that in several cases stone and slag beds have formed units of the same plant, with the opportunity to compare the behavior of each.

The Masonry Committee of the Amer. Railway Engineering Association[12] has also recorded its inability to find instances of corrosion of steel in slag concrete:

> The extensive use of slag concrete over a period of years has demonstrated its permanence for buildings, retaining walls, bridges and foundation work where the structure is exposed to ordinary conditions. No case has been brought to notice where embedded steel became corroded in slag concrete.

The U. S. Bureau of Standards[1] has conducted tests to determine whether there is any corrosive effect of slag, and made the following report:

> To the extent of the tests there was no sign of disintegration due to sulphide sulphur or other causes. In the past there has been some hesitation on the part of engineers to permit the use of slag aggregate without certain special limitations on the material such as weight per cu. ft. and sulphur content. Experience has shown, however, that there is probably no greater need for such restrictions in the case of blast furnace slag than for other commonly used and accepted aggregates.

Sanford E. Thompson[14] made an investigation of slag concrete structures in a number of cities in order to determine the durability of the material, and reported:

I have made chemical and physical tests and also visited several cities where slag concrete has been used for a number of years. As a result of careful examination of various structures made with slag concrete, and interviews with users of slag concrete, I find no evidence of disintegration where portland cement was used. In certain cases with puzzolan cement, the concrete is not in first-class condition, but this is to be expected with any puzzolan cement concrete laid in air. No authentic cases of deterioration of slag concrete made with portland cement, or of rusting of steel embedded in such concrete, have been discovered.

Tests at the Pittsburgh Testing Laboratory[15] showed no corrosion of steel in 5-year old concrete made with slag aggregate from 9 sources in the United States. Slight rust spots were found where air pockets had formed, but these were common to all the aggregates used in the tests—limestone, dolomite, trap-rock, granite, gravel and blast furnace slag.

A. T. Goldbeck[19] is one of the many concrete authorities who have been unable to find a case of slag concrete corroding steel reinforcing, and states:

I know of no cases of well-defined corrosive action of reinforcing steel in slag concrete. I examined, some 8 years ago sections of reinforcing steel removed from a concrete slab in our presence at the Swedeland (slag) plant in Pennsylvania. The sections removed were about three feet long. Two of them were perfectly sound, one end of the third rod had a few pits, showing no rust, but the surfaces were perfectly bright. Since the slab removed had been situated between electric railroad tracks and since there was considerable moisture present, I have always felt that this pitting might have been due to electrolytic action. We were told that this slab had been constructed 11 years previous to the time of our inspection.

Other statements regarding the durability of slag concrete have been made by Wilbur J. Watson, consulting engineer, Cleveland, Ohio[17]; A. D. Bender of Cleveland Macadam Co.[18]; John M. Goehring of Duquesne Slag Products[18]; L. W. Walter of the Erie Railroad[20]; F. Tschudy of the Tennessee Coal, Iron & R. R. Co.[18,21]; C. S. Hill of Eng.-News Record[22]; and W. S. Harger, of the New York State Highway Dept.[23].

Resistance to High Temperatures and Fire

When portland cement concrete is subjected to high temperatures such as commonly occur in building fires, the type of aggre-

gate used may have a decided influence on the resistance of the concrete to deterioration. Blast furnace slag experiences no disruptive change of volume under such conditions because of its low and uniform thermal expansion at all temperatures.

One of the earliest fire tests on record was that conducted by the British Fire Prevention Committee[24] in 1906, consisting of a fire endurance test followed by a hose stream test, on a loaded floor. The slag concrete bay in the floor was reported as showing slightly less buckling and much less cracking than other bays containing gravel, granite, clinker and brick as aggregates. Only the bay containing coke breeze as aggregate was in better condition.

Later tests by the British Fire Prevention Committee[25] in 1917-19 included as aggregates, two blast furnace slags, six gravels, seven sandstones, four limestones, eleven granites, five bricks, two coke breezes, two pan breezes and three coal clinkers, in both plain and reinforced concrete slabs. Fire and hose stream tests were made on the loaded slabs, and thermal conductivity values determined. The report of the tests states:

> Coarse aggregates of clean broken bricks and burnt clays gave the most satisfactory results from strength and resistance to fire. Blast furnace slag appeared to be almost as good. Pan breeze and clinker were fairly satisfactory. The gravels, sandstones, granites and other highly siliceous aggregates were unsatisfactory, while the fine-grained igneous rocks, such as basalt, gave satisfactory results. The limestones were better than the siliceous aggregates. In parallel with the tests of plain and reinforced concrete slabs, the heat conductivity of the various concretes was measured. The coarse aggregates in the slabs which did not attain to 1200° F. were slag, limestone, basalt, andesite, coke breeze, broken brick and whinstone. The highest temperatures at points near the top of the slabs were attained with concretes having as coarse aggregates siliceous gravels, calcareous gravel, coke breeze and quartzite.

W. A. Hull[26] reports fire tests on loaded concrete columns which were later tested in compression. Part of his conclusions follows:

> The results contribute additional evidence that gravels of 3 distinct types, all high in quartz, give less satisfactory results under fire conditions than concretes from limestone, traprock and blast furnace slag aggregates. Columns of these types from traprock and blast furnace slag concrete show no tendency to spall or crack to any important extent under the conditions of the standard 4-hour fire test. In all cases, columns from traprock and slag aggregates have

shown themselves capable of bearing considerably more than twice their respective working loads before cooling, following the 4-hour fire test. The observation that neither the traprock nor the slag concrete appears to have any tendency to spall or any other malignant tendency under the conditions of these tests is important and reassuring.

Fire tests of concrete building units at the Underwriters' Laboratories[27] in Chicago resulted in the conclusion that "Coarse aggregates of limestone, calcareous pebbles, cinders or crushed slag perform with comparative similarity under standard fire exposure conditions."

Tests of German concrete aggregates by Dr. Kurt Endell[28] included blast furnace slag, granite, basalt, limestone, quartz, quartzose and burnt brick, and resulted in the following report:

Quartz and granite expand up to 930° F. and then undergo an alteration which is accompanied by cracking of the material. Basalt and blast furnace slag show a smaller expansion due to heat and one which proceeds uniformly. The practical results of the investigation show that for heat-resisting structures quartz-containing mixtures should not be used, but aggregates composed of blast furnace slag, basalt and limestone should be employed because of their slow and uniform expansion.

In October, 1925, at Birmingham, Ala., a small house built of slag concrete tile was filled with inflammable materials and ignited. While the walls were white hot a hose stream was turned on it, but the State Fire Marshall, Chester E. Johnson[29] reports:

On inspection of the building no cracks of any nature were found, nor was any disintegration or crumbling of the tile apparent. On breaking through the wall and examining the interior webs of the tile, no cracks or other evidence of failure were found. The ceiling was undamaged, and the concrete tile roof showed no evidence of having been affected. The units plainly demonstrated their fire-resisting qualities and are to be highly recommended for use in the construction of permanent and fireproof structures.

Exactly 1 year later the same type of tile underwent a severe test of its fire-resisting properties in actual construction. During the building of the Calvary Baptist Church in Tuscaloosa, Ala., using slag concrete tile, the structure was destroyed by fire except for the tile, which withstood the flames in an extraordinary manner. Quoting from the report in *Concrete:*[30]

Fortunately for the underwriters and the congregation, the concrete tile walls resisted the flames in a remarkable manner. The blaze gained rapidly, probably having made considerable headway before discovery. Then

for two and a half hours the structure burned fiercely, consuming timbered and wooden interior, scaffolding, floors and finally the roof, which was supported by unprotected steel trusses. The walls were relatively quite high for their length, but this fact has no bearing except to accentuate their good behavior in the fire. There were five large arched windows in each side, as well as other large openings which provided plenty of draft and tended to weaken the walls. When the fire was finally subdued . . . two of the main walls had been carried down. These walls were forced over by the collapse of the roof trusses after having been deprived of the bracing effect of the previously collapsed floor members. The high front wall, one of the sidewalls, and the chimney remained standing. The chimney will doubtless be used in rebuilding the structure. The stucco apparently is in perfect condition except where blackened by smoke, although subject to the very severe test of having water from the fire hose thrown upon it while in an extremely hot condition. There was no discernible distortion of these walls at any point, a rather remarkable fact under the circumstances. No cracking or spalling of the units was found and the walls which were overturned broke up into a few large pieces of masonry apparently as strong as any first-class masonry not exposed to fire. As a result of the fire performance of the concrete tile in this fire, the same material was purchased for the rebuilding of the structure.

George H. Sanger[31] cites the use of slag concrete at the plant of the Northwestern Iron Co. at Mayville, Wis., as an example of its resistance to high temperatures:

In the last five years we have placed nearly 30,000 yards of slag concrete for various purposes and subject to a wide range of conditions and I have never known of a failure that could in any way be traced to the slag. In places where the concrete has been subjected to a considerable amount of heat, some few cracks have developed, but they were undoubtedly due to the expansion.

Our cooling pit at the pig machine probably represents the most severe conditions imposed upon any of our concrete. The pit is first filled with cold water, into which the pig machine discharges red-hot pig iron, which soon heats the water to boiling point; at the end of the cast the hot water is drawn off and the pit is refilled with cold water. As this cycle occurs every two hours the concrete is in a continual state of expansion and contraction, and is also in direct contact with the boiling water and steam. Thus far there have been no signs of spalling or surface cracks; the surface is apparently in as good condition as it was when the forms were first removed.

Compressive Strength

More data have been compiled on the compressive strength of concrete than on any other quality, although recently there has been a tendency to use other specific tests along with the compressive where the concrete is to undergo a particular type of

service. For example, pavement concrete is tested for transverse strength as well as compressive. However, it is believed by most concrete authorities that the compressive strength of a concrete is a very fair general indication of all-round quality.

Other conditions of test being equal, such as cement content, water-cement-ratio, etc., there is very little difference between the compressive strengths of concretes employing the more common mineral aggregates. This is illustrated in the results of tests at the laboratory of the Portland Cement Association[32], which included as aggregates 19 slags, 27 gravels, 36 limestones, 14 granites, 11 sandstones and 14 traprocks. Since each aggregate was selected from a different source, these tests should furnish a representative picture of the relative compressive strength of slag concrete. The mix was 1 part of cement to 4 parts of dry rodded mixed aggregates, which resulted in practically equal cement content throughout. Two combinations of aggregates were used: "A" containing 38 per cent sand and "B" 48 per cent. The coarse aggregates were graded differently in each of the combinations: "A" having 25 per cent No. 4-⅜ in., 50 per cent ⅜-¾ in., and 25 per cent ¾-1½ in., while the "B" grading was 25 per cent ⅜-¾ in. and 75 per cent ¾-1½ in. The cylinders were stored 14 days in the moist room, then in air till tested at 90 days, 5 cylinders being broken for each concrete containing a different aggregate. In the "A" grading, the average of all concretes was 4393 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were as follows: Limestone—103; Sandstone—101; Granite—99; Gravel—98; Slag—97; and Traprock—96. In the "B" grading, the general average was 4217 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Limestone—102; Traprock—101; Granite—99; Slag—98; Gravel—97; and Sandstone—96. It is apparent that the difference between aggregates is not sufficient to justify precedence of one over the other.

Stanton Walker[33] has reported other tests at the P. C. A. Laboratory on 7 different mixes varying from 1:2 to 1:6, the proportioning being on the same basis as before. Limestone,

granite, gravel and slag were the aggregates tested, and two curing methods were used: (a) In damp sand for 90 days, and (b) In damp sand for 14 days and the remainder of 90 days in air. In the damp sand curing, the average value for all mixes and all aggregates was 3940 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Granite —103; Limestone—100; Slag—98; and Gravel—96. In the air curing, the general average was 3626 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Limestone—102; Granite—101; Gravel—99; and Slag—97. Here again the difference between aggregates is so slight as to be negligible.

Not quite so close an approximation of compressive strengths is noted in further tests at the P. C. A. Laboratory reported by Duff A. Abrams[34]. The aggregates were slag, limestone, gravel and granite, tested at 1 year after storage in damp sand. The proportions were 1 part cement to 4 parts of dry rodded mixed aggregates. The general average was 4312 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Gravel—115; Slag—105; Limestone—91; and Granite—87.

Tests of the U. S. Bureau of Roads recently reported by W. F. Kellerman[35] included as aggregates 3 slags, 4 limestones, 1 sandstone, 1 traprock, 1 granite, and 7 gravels, in 4 distinct gradations. Two mixes were used, 1:1.6:3 and 1:2:4, based on dry rodded volumes of aggregates. Tested at 28 days after storage in moist air, the grand average strength for the richer mix was 3682 lb. per sq. in. Taking this value as 100,the averages of the individual aggregates were: Sandstone—113; Slag—101; Granite—99; Limestone—98; Gravel—97; Traprock—92. The grand average strength for the leaner mix was 2776 lb. per sq. in. Taking this value as 100, the average of the individual aggregates were: Sandstone—113; Limestone—100; Gravel—99; Granite—99; Slag—96; Traprock—94.

Raymond Harsch's[4] investigation for the U. S. Bureau of Roads included compression tests on slag concrete specimens. These were made with slags selected from 19 different sources in the United States, the mix being proportioned 1:2:3 based on dry rodded volumes of aggregates. For check aggregates one

gravel (average quality) and one limestone (above average quality) were used. Tested after 28 days damp sand storage, the average of all concretes was 3123 lb. per sq. in., 3-5 cylinders being tested for each aggregate source. Taking this average as 100, the slag concretes averaged 104, the stone concrete 95 and the gravel concrete 85.

Nine slags from as many sources were included in P. J. Freeman's[15] ten-year concrete tests, as well as two gravels, one traprock, one granite, and two limestones. The mix was proportioned 1:2:4 based on dry compact volumes of aggregates, and compression cylinders were broken at various ages up to 10 years. The strengths of all concretes were affected similarly by the type of storage, which was 35 days in damp sand followed by the remainder of 5 years in a warm dry laboratory. At 5 years, the cylinders were stored outside in the weather until 9⅔ years, when they were placed in damp sand until tested at 10 years. Mr. Freeman states in his report:

> All concrete, regardless of type of coarse aggregate, increased in strength when stored in dry air up to the age of 1 year, and after that time decreased in strength so long as the air storage continued. The strength of the concrete was regained after storage in the weather, and at the end of 10 years it has exceeded in strength the results obtained at 1 year.

Mr. Freeman's report also states that the compressive strength was not affected by:

> (a) Weight of the slag aggregate. The compact weight varied from 64 to 84 lb. per cu. ft.
>
> (b) Chemical analyses of the slags. The silica content varied from 31.53 to 36.72 per cent; the alumina from 11.81 to 14.11 per cent; the lime from 31.75 to 45.80 per cent; and the magnesia from 1.62 to 19.43 per cent.
>
> (c) Age of the slags. Some were only a few days old when used, while others had lain in banks for as much as 15 years.

At the end of 5 years in warm dry air, the general average of all concretes was 3939 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Slag—107; Granite—102; Traprock—94; Limestone—92; and Gravel—77, these values being the average of 27 specimens of slag concrete, 6 of gravel, 3 of traprock, 3 of granite and 6 of limestone. At 10 years, the general average strength was 5616 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Granite—103; Limestone—102; Slag—100; Trap-

rock—95; and Gravel—90. The values at 10 years were the average of nine specimens of slag concrete, five of limestone, two of gravel, two of granite and one of traprock.

Transverse Strength

The earlier beam tests of concrete made in a compression machine, consisted of supporting the beam at each end and applying the load either at the center point of the top surface or at the third points. Third-point loading was used in the P. C. A. Laboratory tests reported by Duff A. Abrams[34], the aggregates being slag, limestone, gravel and granite. The proportions were 1 part cement to 4 parts dry rodded mixed aggregates, which resulted in practically equal cement content throughout. At 7 days, the general average of all concretes (7 x 10 x 36 in. beams) was 423 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Slag—106; Limestone—104; Gravel—99; and Granite—90. At 28 days, the general average was 557 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Limestone —106; Slag—104; Gravel—98; and Granite—97. The foregoing figures are based on the average of 5 breaks, except the 28 day gravel value, which is on 25 breaks.

Later tests at the P. C. A. Laboratory used the same mix, method of test, and aggregates except that sandstone was included. These were reported by H. F. Gonnerman and E. C. Shuman[36]. At 7 days, the average strength for all concretes (7 x 10 x 38 in. beams) was 454 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Limestone—112; Sandstone—101; Slag—99; Gravel—98; and Granite —90. At 28 days, the average of all concretes was 609 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Limestone—115; Slag—102; Gravel—98; Granite—95; and Sandstone—92. These values are based on an average of 5 breaks, except the 28-day gravel value, which is on 15 breaks.

Center-loading tests of slag concrete have been reported by F. H. Jackson and J. T. Pauls[37] in connection with the Arlington Circular Track experiment of the U. S. Bureau of Roads. These were on 6 x 8 x 48 in. beams tested at 90 days in a 1:1½:3 field mix. The aggregates were 9 gravels, 10 stones and 3 slags. The

average of 2 beams for each aggregate showed a general average of 575 lb. per sq. in. Taking this value as 100, the average of the slag concretes is represented by 117, the stone concrete by 99 and the gravel concrete by 95.

During the construction of a 5½-mile section of the Cleveland-Meadville Road[38] in 1928, slag concrete beams were tested daily by the cantilever test at every age from 3 to 16 days, and thereafter intermittently until 26 days. The mix was 1 part cement to 5 parts of total aggregates (fine and coarse) measured loose, and the beams were 6 x 6 x 40 in. The machine used was a portable one developed by the Ohio Highway Dept., using a worm-and-gear for applying the load and a steel extension arm. The 3-day strength was 598 lb. per sq. in. Taking this value as 100, the average strengths at other ages were as follows:

Four day—110; 5—110; 6—111; 7—117; 8—129; 9—124; 10—125; 11—116; 12—127; 13—139; 14—126; 15—122; and 16 day—130. At 26 days the strength was 817 lb. per sq. in., or an increase of 36 per cent over the 3-day strength.

The average of all slag concrete pavements built by the Ohio Highway Dept. in 1928 shows the transverse values tabulated below. These data have been furnished from the files of the department through the courtesy of A. S. Rea and D. W. Leggett[39], and represent 370 tests of 6 x 6 x 40 in. beams. The mix throughout was the same as shown for the Cleveland-Meadville Road:

AGE AT TEST DAYS	NUMBER OF BEAMS	AVERAGE TRANSVERSE STRENGTH
3	1	598
4	4	653
5	7	655
6	4	611
7	102	656
8	11	696
9	8	734
10	7	675
11	4	675
12	10	715
13	4	766
14	102	732
15	2	805

16	2	743
21	17	762
28	85	801

For concrete base work the Ohio Highway Dept. uses a leaner mix of 1 part cement to 7 parts total aggregate (fine and coarse) measured loose. The average of 107 slag concrete beams tested during 1928 showed strengths of 446 lb. per sq. in. at 7 days, 544 lb. at 14 days, 641 lb. at 21 days, and 602 lb. at 28 days.

Tests at the Ohio Highway Dept. Laboratory in 1928 reported by A. S. Rea[40] indicate that the type of testing machine, or the type of specimen, or both, affects the results appreciably. The machine used by the Ohio Highway Dept. takes a 6 x 6 x 40 in. specimen, while the original machine developed by H. F. Clemmer and Fred Burggraf[41] takes a 6 x 8 x 30 in. beam. Five slags from as many sources were tested in 2 mixes, 1:5 and 1:5½, proportioned 1 part cement to a stated number of parts of total aggregate measured in a dry loose condition (cement content respectively 6.7 and 6.4 sacks per cu. yd. for the two mixes used). The five slags were chosen to represent the extremes found in commercial practice, ranging from 61 to 89 lb. per cu ft. (dry, loose weight). The weight per cu. ft. did not seem to be reflected in the transverse strength, since the heaviest slag had only a slight advantage in strength at 7 days and this disappeared at later ages. The results of the tests are shown in the following tabulation:

MIX	AGE AT TEST, DAYS	AVERAGE STRENGTH OF SLAG CONCRETES, MODULUS OF RUPTURE	
		Ohio Machine	Illinois Machine
1:5	7	672	548
	28	761	623
	90	861	709
1:5½	7	674	556
	28	744	607
	90	818	675

Slag concrete pavements built by the Pennsylvania Highway Dept. in 1928 were proportioned by either weight or volume. The average of 1312 beam tests of slag concrete, 6470 of stone concrete, and 618 of gravel concrete are shown in the accompanying tabulation reported by H. S. Mattimore[42]. The tests

were made on a machine using a wood extension arm, the specimens being 6 x 8 x 40 in.:

MIX	AGE AT TEST, DAYS	AVERAGE STRENGTH, MODULUS OF RUPTURE Slag	Stone	Gravel
*1:2:3	10	633	583	576
	14	656	631	596
	21	681	601	—
†1:2:3¼	10	617	589	477
	14	621	612	591
	21	726	—	—
‡1:2:3½	10	634	543	476
	14	627	593	534
1:2:4	10	—	434	497
	14	596	474	528
	21	665	559	545

*By volume, with no compensation for bulking of sand
†By volume, with compensation for bulking of sand.
‡By weight, with compensation for moisture in sand.

The U. S. Bureau of Roads tests reported by W. F. Kellermann[35] were also made with the machine having the wood extension arm, the load being applied through a worm and gear arrangement. Four limestones, 1 granite, 1 sandstone, 1 traprock, 3 slags and 7 gravels were the aggregates in two mixes,—1:1.6:3 and 1:2:4, based on dry rodded volumes of aggregates. The three slags varied in compact weight from 65 to 82 lb. per cu. ft., but this weight variation could not be traced in the transverse test results. Each coarse aggregate was tested in 4 different gradings, but the gradations as used seemed to have no effect on the transverse strength. At 28 days the average of all concretes in all gradings in the 1:1.6:3 mix showed a value of 599 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Slag—106; Limestone—100; Trap—99; Granite—98; Gravel—96; and Sandstone—93. In the 1:2:4 mix at 28 days, the general average for all gradings was 502 lb. per sq. in. Taking this value as 100, the averages of the individual aggregates were: Limestone—104; Sandstone—103; Slag—102; Gravel—97; Trap—95 and Granite—95.

Resistance to Wear

The ability of pavement concrete to resist wear is not of primary importance, since no concrete road has ever become unsatisfactory purely from attrition. Since the wear on a concrete road

surface is very small, the pavement will have failed from other causes long before the wear can become a factor in its deterioration. The best-known laboratory test for wear of concrete is the Talbot-Jones, in which the inner circumference of a steel cylinder is lined with 8 x 8 x 5 in. concrete blocks and revolved 900 times in each direction after the addition of an abrasive charge of cast iron balls weighing 200 lb. Duff A. Abrams[43] reports the results of Talbot-Jones tests on concretes containing 13 slags, 24 gravels, 26 limestones, 12 granites, 6 traprocks, 7 sandstones, 2 flints and 1 marble, proportioned 1 part cement to 4 parts of dry rodded mixed aggregates (approximately equal cement content throughout). The average depth of wear, based on 10 specimens of each aggregate sample, showed the lowest value for granite concretes,—0.34 in. The average values for the other concretes were as follows: Traprock—0.40 in.; Marble—0.42 in.; Limestone—0.44 in.; Sandstone—0.45 in.; Flint—0.46 in.; Slag—0.48 in.; and Gravel—0.51 in. In later tests[44] designed to show the effect of cement content upon wearing resistance 8 mixes were used varying from 1:1 to 1:6, proportioned as above. Granite concrete showed the least average wear for all mixes,—0.44 in., followed by slag with 0.49 in., limestone with 0.51 in., and gravel with 0.54 in.

In the Circular Track Experiment of the U. S. Bureau of Roads[37] the Talbot-Jones test was used as a laboratory check on the wearing resistance of concrete which later was subjected to the actual service test of chain-tire traffic. The three slag concretes showed an average depth of wear in the Talbot-Jones machine of 0.25 in., the 10 limestone concretes 0.28 in., and the nine gravel concretes 0.30 in. The actual track test consisted of running two solid-rubber-tired truck wheels over sections of the various concretes. The wheels were loaded to 6,000 lb. and run over the track 55,000 times, but the wear from this operation was negligible so the wheels were equipped with tire chains and an additional 25,000 runs made. The average depth of wear for the nine gravel concretes was 0.31 in., for the three slag concretes 0.37 in., and for the 10 stone concretes 0.46 in. The best slag, with a wear of 0.27 in., made about the same showing as the best stone and gravel concretes. That this test was more severe than actual

traffic conditions is noted in the conclusions of the report, where it is stated:

It is realized that chain traffic on concrete highways is not dense enough except in isolated instances to cause any such wear of the surface of a pavement as was produced on the test sections. Wear, however, does take place at the unprotected edges of joints and cracks. It was believed that the relative resistance offered by the various sections to the chain traffic would be a fairly reliable index of the ability of the materials to resist the combined destructive influences of traffic, whether such influences produce wear at the joints or cracks or cause surface disintegration.

Toughness

Toughness in concrete is essential for whatever purpose it is to be used; buildings must withstand vibration and shocks, and pavements must be able to stand up under the impact of traffic. H. S. Mattimore[45] reports the results of impact tests on concretes containing slag, traprock, limestone, sandstone, slate and gravel, in 3 volumetric mixes,—1:1½:3, 1:2:3 and 1:2:4. The test machine consisted of a striking head, mounted on a frame, which was raised and released by a cam. The striking points were horse shoe calks with springs fitted to them to give ½ in. play so as to allow for wear of the matrix between the aggregate. The specimen was rotated by a dog attachment so that successive blows would be delivered to different parts of the surface. The effective blow was 29 lb. dropping 4 in. at the rate of 90 per minute.

The molded cylinders used in the test were made from the concrete going into the Lancaster Test Road built by the Pennsylvania Highway Dept. in 1919-20. Cores were also drilled from the pavement after 8-15 months. When tested under 5000 blows of the machine, the average loss in weight of the various specimens was as represented by the values in the following tabulation, which are expressed as "impact coefficients," a high coefficient indicating a high loss, and vice versa:

AGGREGATE	NUMBER OF CORES	1:1½:3 MIX		1:2:3 MIX		1:2:4 MIX	
		Drilled	Molded	Drilled	Molded	Drilled	Molded
Sandstone	35	115	114	115	140	90	82
Limestone	46	116	138	97	140	91	104
Slag	22	142	211	146	148	149	153
Gravel	14	191	161	168	152	212	165
Slate	16	191	193	145	134	156	137
Traprock	6	—	—	—	200	—	150

A summary of the foregoing tabulation for all mixes and both conditions of test shows an average coefficient of 133 for all concretes. Taking this value as 100, the averages of the individual aggregates were: Sandstone—83; Limestone—84; Slate —115; Slag—117; and Gravel—130, indicating an intermediate toughness for slag concrete. However, Prof. Curtis C. Myers'[72] tests at University of Cincinnati showed slag concrete tougher than either limestone or gravel concrete in a 1:2:4 mix. Five-inch cubes of concrete were first tested in the impact machine, which consisted of a swinging weight striking a blow equal to a 25-lb. weight falling 15 in., the load being concentrated on a 1-in. diameter striking point. The gravel concrete withstood 6 blows, the stone concrete 7 blows, and the slag concrete 10 blows before failure. Concrete beams were then tested in the machine using slag and stone as aggregates, and increasing the height of the blow each time. The stone concrete absorbed 8 blows before failure against 12 for the slag concrete, each of the last 4 on the slag concrete being more severe than that which caused failure of the stone concrete.

George W. Hutchinson[46] reports impact tests on limestone, slag and gravel concretes of equal cement content (6¾ sacks per cu. yd.) in which the testing machine consisted of a 10-lb. weight falling perpendicularly on the specimen from a constant height. The specimen was a 20-in. concrete disk 6 in. in diameter, supported on 3-point rubber-padded bearings, and the shock was transmitted through a 3-in. steel ball resting on the center of the top surface. The average of several hundred tests showed that limestone concrete had withstood 138 blows before failure, slag concrete 100 blows and gravel concrete 62 blows, but the author points out that there was equally as much variation within one type of aggregate as there was between different types.

Workability and Yield

Workability is defined as that quality in concrete which provides ease in mixing and handling, and allows the mixture to be readily placed without tangible voids, honeycombing or segregation.

Slag in common with other crushed aggregates, tends to give harsher mixtures than rounded aggregates because of its rougher

surface and higher voids, but this tendency can be largely overcome by attention to proper grading, and the use of a somewhat higher ratio of fine to coarse aggregate than would be required for rounded aggregates. This so-called "over-sanding" has been proven beneficial in every way, according to tests at the U. S. Bureau of Standards[47] with slag, limestone and gravel aggregates. The conclusions state:

> The tests showed (1) That for the various combinations of fine and coarse aggregates the amount of sand required to give a strength at least as great as that indicated by the normal water-cement-ratio strength-relation varied from about 33 to about 50 per cent of the total aggregate, (2) That the percentage of sand as determined gave a satisfactory workable mix, (3) That it was about the smallest percentage of sand which would give a workable mix, and (4) That these same percentages of sand gave higher strengths per barrel of cement per cu. yd. of concrete than did any other combinations of the same aggregates; that is, they gave approximately the maximum economy of cement.

The above conclusions are of great importance to the user of crushed aggregates, since they indicate that "oversanding" not only gives a workable mix but actually shows maximum economy in cement. This point can not be too highly stressed, because in the past arbitrary proportions of 1:1½:3, 1:2:4 etc. have been used with crushed aggregates in many cases where slightly more sand and less coarse aggregate would have greatly increased the workability, at the same time slightly increasing the strength and yield. This is brought out very forcibly in earlier experiments at the Bureau of Standards[48] reported in 1920. A limestone concrete in a 1:2:4 mixture was found to be harsh and segregating, while a 1:3:3 mix of the same materials not only gave satisfactory workability but higher 28-day strength, in spite of the fact that less cement was used per cu. yd. of concrete. Further tests showed that all mixes with the same proportion of cement to total aggregates had substantially the same strength, and that the oversanded mixes consistently used less cement.

Yield is the volume of concrete resulting from given quantities of cement and aggregates, and may be expressed in the following different ways:

1. The ratio of the volume of finished concrete to the volume of mixed aggregates, dry and rodded. Thus if a mix of 1 sack of cement and 4 cu. ft. of rodded mixed aggregates results in 4.4 cu. ft. of concrete, the yield is 1.10, or 110 per cent.

2. The ratio of the volume of finished concrete to the volume of the coarse aggregate, the latter being measured in any definite way. Thus if a mix of 1 sack of cement, 2 cu. ft. of sand, damp and loose, and 3 cu. ft. of coarse aggregate, damp and loose, results in 3.9 cu. ft. of concrete, the yield under these conditions is 1.30 or 130 per cent of the volume of the coarse aggregate as measured.

3. The volume of concrete of any proportions produced by one sack of cement. This is more often expressed as the reciprocal measure, viz: the number of sacks of cement contained in a cubic yard of concrete, and this number divided by 4 (to convert to barrels) is commonly referred to as the "cement factor." Thus if 6.4 sacks of cement are required for a cubic yard of concrete, the cement-factor is 1.6.

This last-mentioned method of expressing yield is now the one most commonly used, and it has the advantage of indicating the type or quality of concrete more definitely than the time-honored designation: 1:—:—, or parts of cement to parts of fine aggregate to parts of coarse aggregate. This fact is worth remembering in that recent discussions of the merits of different types of coarse aggregate have emphasized the necessity of comparing concrete mixtures on the basis of equal cement content, and when this is done with heed to the proper balance between fine and coarse aggregate for workability, the differences in concrete making properties of the different coarse aggregates become insignificant. Unfortunately this basis of equal cement content in comparative tests has not been generally used up to this time and most of the test data quoted in this report are of necessity taken from tests based on like proportions of cement and aggregates by volume or by weight.

Weight proportioning was used by Fred Hubbard[50] in comparative yield tests which he has reported using slag, limestones, traprock, gneiss and gravel as aggregates. The materials were proportioned on the basis of their respective dry weights, corresponding to a volumetric proportion of approximately 1:2:3 field mix. The gravel concrete showed the lowest cement factor, —6.80 sacks per cu. yd. of concrete, followed in order by gneiss with 6.91, traprock with 7.16, limestone No. 1 with 7.28, slag with 7.32 and limestone No. 2 with 7.32. In a leaner proportion

corresponding to approximately 1:2½:3½ field mix, limestone No. 2 and slag again had the same cement factor,—6.31 sacks per cu. yd. of concrete.

Slag as Fine Aggregate

Many more or less successful attempts have been made to introduce the finer sizes of slag (both air-cooled and granulated) as a substitute for natural sand in concrete, and a number of strong and durable structures have been and are being built in this way. Two prejudices had to be overcome before slag could be used as fine aggregate: First, the natural reluctance to change from a known to a practically unknown material; and Second, the fact that the resulting mix is somewhat lacking in workability. Laboratory and service tests have removed all doubt as to the strength and durability of concrete made with slag fine aggregate, while the workability of such a mix can be greatly improved by the addition of a portion of fine sand or a small amount of powdered slag, diatomaceous earth, hydrated lime, etc.

The control tests for the Circular Track investigation of the U. S. Bureau of Roads[37] showed satisfactory results for an all-slag concrete in compressive strength, transverse strength and resistance to wear. The transverse tests were made in the compression machine with center-loading, and the wear tests in the Talbot-Jones machine. Nine gravel concretes, 10 stone concretes and 4 slag concretes made with natural sand were included in these tests, as well as 1 all-slag concrete made with air-cooled slag screenings as fine aggregate. Taking the average of all concretes as 100 in each case, the relative values of the individual concretes in a 1:1½:3 field mix at 90 days were as shown in the accompanying table. With one exception, the all-slag concrete showed higher strengths and less wear than the concretes made with natural sand as fine aggregate:

KIND OF CONCRETE	COMPRESSION STRENGTH	TRANSVERSE STRENGTH	WEAR TEST
All-slag	104	133	68
Average of 3 slag concretes	101	112	88
Average of 10 stone concretes	105	97	97
Average of 9 gravel concretes	93	93	104

With reference to the foregoing table it should be stated here that the relation indicated by the laboratory wear tests was not borne out in the actual test on the track. The all-slag concrete showed more wear under tire-chain traffic than all the other concretes with the exception of one slag concrete and two stone concretes, which led F. H. Jackson and J. T. Pauls to state in their conclusions from the tests that "Slag or stone screenings are, in general, unsatisfactory as substitutes for natural sand as fine aggregates in concrete road construction."

Other tests on slag as fine aggregate have been reported by Curtis C. Myers[2] who found that an all-slag concrete withstood impact better than other concretes made with natural sand: and by Duff A. Abrams[52] who found that gravel concrete made with granite, slag, limestone and traprock screenings as fine aggregates were somewhat weaker (in compressive strength) than sand-gravel concrete.

The use of slag as fine aggregate has been largely confined to concrete construction around the steel plants. Fifty all-slag concrete houses were built at Youngstown, Ohio, in 1919 by the Carnegie Steel Co., and similar construction was used in recent additions to slag plants at Youngstown, East Toledo and Massillon, O. The Ford Hotel at Buffalo, N. Y., was built with an all slag concrete in 1923, and in 1916 four slag concrete bridges were built at Pottsville and St. Clair, Pa., using as fine aggregate equal parts of natural sand and slag sand.

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Readers are referred to the JOURNAL *for February, 1931, for discussion which may develop. Such discussion should reach the Secretary by January 1, 1931.*

Discussion of Progress Report of Committee 105

"Reinforced Concrete Column Investigation"*

BY HERBERT J. GILKEY†

A RECENT paper by Richart and Jensen[1] reports that Haydite concrete having an ultimate strength and bond resistance about equal to those of concretes from ordinary aggregates, has a modulus of elasticity that is decidedly lower. This fact means that the values of "n" are increased and that a correspondingly higher stress can be transferred to the steel for a given stress in the concrete.

From this very interesting fact it appears that Haydite concrete may have unique possibilities in the realm of columns and other reinforced compressive members. This point has been enlarged upon in a discussion of the Richart and Jensen paper[1].

It is probably too late to inject any phase of Haydite concrete into the present project even if it were deemed desirable to do so. It does appear that this aspect of the material has enough novelty and promise in it to warrant some sort of a separate experimental reconnaissance over the field.

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*JOURNAL A. C. I., April, 1930; *Proc. A. C. I.*, Vol. 26, p. 601.

†Professor of civil engineering, University of Colorado, Boulder, Colo.

[1]Proc. A. S. T. M., 1930, Part II.

Discussion of Paper by P. M. Woodworth

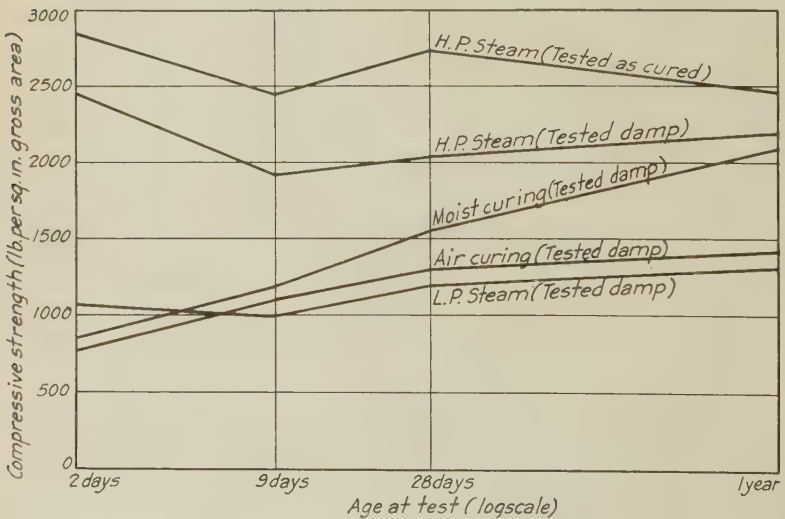
"Some Tests of Concrete Masonry Cured With High-Pressure Steam"*

AUTHOR'S CLOSURE

This investigation has been completed with the testing of the one-year specimens. A summary of the results is contained in the accompanying table A and typical age-strength relations are plotted in the figure.

The outstanding feature of these tests is that concrete units cured with steam under 100-lb. pressure for 12 hr. and then in air at 50 per cent relative humidity, developed strengths which averaged about 15 per cent more at 2 days and about 5 per cent

*A. C. I. Journal, Feb. 1930, *Proceedings*, Vol. 26, p. 504.



Effect of curing on compressive strength of concrete masonry units

SOME TESTS OF CONCRETE MASONRY UNITS CURED WITH HIGH PRESSURE STEAM

TABLE A—*Compressive Strength of Vibrated Concrete Masonry Units*

Compressive tests of 5 by 3¾ by 12-in. concrete tile.

Cement from fresh warehouse stock.

Aggregate: Crume Brick Co. bank sand and crushed gravel graded 0-⅜-in.

Water: 8½ gal. per sack corrected for absorption of aggregate.

Mix: 1:4:6 damp, loose volume; 1:9.6 dry weight.

Concrete vibrated in molds for 50 seconds at 3600 r. p. m.

Specimens removed from molds 36 to 48 hr. after molding and air-cured until test.

Specimens capped with gypsum 6 hr. before testing.

Unless otherwise noted each value is the average of three tests made on the same day.

Values in parentheses are percentages of 28-day strengths of moist-cured tile.

Age Before Curing Hr.	Method of Curing	Condition When Tested	Compressive Strength lb. per sq. in. Gross Area			
			2-Da.*	9-Da.	28-Da.	1-Yr.
½	12½ hr. in saturated steam at 100 lb. gage pressure then air-cured at 70° F.	Damp	2250 (144)	1820 (117)	1890 (121)	2150 (137)
		As Cured	2900 (186)	2500 (160)	2550 (164)	2490 (160)
4	"	Damp	2440 (157)	2020 (131)	2060 (132)	2250 (144)
		As Cured	2960 (190)	2560 (164)	2620 (166)	2620 (168)
8	"	Damp	2530 (161)	1970 (126)	2150 (133)	2420 (155)
		As Cured	2760 (177)	2580 (165)	2990 (192)	2300 (147)
24	"	Damp	2560 (164)	1880 (121)	1990 (128)	1940 (124)
		As Cured	2840 (182)	2140 (137)	2740 (176)	2460 (158)
Average	"	Damp	2450 (157)	1920 (123)	2020 (130)	2190 (140)
		As Cured	2760 (187)	2450 (157)	2720 (174)	2470 (157)
4	12½ hr. in steam at 125° F. and atmospheric pressure then air-cured at 70° F.	Damp	1070 (69)	1000 (64)	1190 (76)	1320 (85)
		As Cured	1150 (74)	1270 (81)	1810 (116)	1430 (92)
6	12½ hr. under damp straw at plant temperature then air-cured at 70° F.	Damp	770 (49)	1110 (71)	1310 (84)	1420 (91)
		As Cured	1060 (68)	1360 (87)	1930 (124)	1570 (101)
0	48 hr. under damp straw at plant temperature then moist - cured at 70° F.	Damp	860 (55)	1190 (76)	1560 (100)	2090 (134)
		Oven Dry				2950 (189)

*2-Day Tests made on 200,000 lb. Testing Machine at Bowser-Morner Testing Laboratories, Dayton, Ohio.

Tests at later ages made on 300,000-lb. Testing Machine at Portland Cement Association, Chicago.

more at 1 year than that of similar units moist-cured for a period of one year. These comparisons are based on units tested in a damp condition.

It appears that high-pressure steam-curing furnishes a means of developing relatively high concrete strengths at very early ages. The promising results obtained by this method of curing show the desirability of further study by manufacturers of concrete masonry units and other concrete products in order that its possibilities may be developed to the greatest extent.

Architectural Concrete—Forms, Molds and Surfaces

BY WILLIAM C. WAGNER*

Architecturally treated, exposed concrete as an exterior finish for buildings is a comparatively recent adaptation of the material, that requires, in addition to purely structural qualities, a smooth, durable finish and a definite sharpness of detail comparable to cut stone or carving. In our efforts to obtain the most satisfactory surfaces with this medium, we have revised many of our former opinions and extended our study to all agents and accessories that contribute to a concrete structure, for the uncertainty of success and the finality of the result make extreme care in preparation a mandatory consideration. Although the work is complicated by the nature of the material which places many limiting conditions on its uses, many of the faults of exposed concrete can either be eliminated or provided for by well studied design.

In building forms for architectural concrete there is no radical departure from the practice accepted for structural concrete, unless it be that they are lined with materials that control the character of the resultant surface. This discussion will therefore be confined to these materials and such variations from usual practice as are required to yield a satisfactory finish. Subsequent remarks may not receive complete concurrence from all who work with exposed concrete, because of such difference of opinion on the various points of procedure as result naturally from individual experience.

In a broad sense there are three types of wall treatment encountered in buildings—plain, ornamented and molded surfaces ——each of which required a different type of form lining.

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FIG. 1—BANK OF BALBOA, BALBOA, CALIFORNIA—MORGAN, WALLS AND CLEMENTS, ARCHITECTS, LOS ANGELES. AN EARLY EXAMPLE OF EXPOSED CONCRETE STRUCTURE EMPLOYING PLASTER WASTE MOLDS AND ORDINARY BOARD FORMS

LINED FORMS FOR PLAIN SURFACES

For plain surfaces, forms are lined with a board of fibrous composition or with built-up wood panels, generally of three-ply construction and for ornamented and molded surfaces plaster casts and wood patterns are used.

To provide a rigid frame for attaching these linings, it is advisable to use 2-in stock. The use of 1-in. stock is questionable economy, because to overcome its tendency to buckle, more extensive bracing is necessary, with consequent additional expense of material and labor.

Fibre board is produced in large sheets which can be readily sawn to size; it is rigid and light in weight and it is easily nailed to forms or drilled to allow the passage of tie wires. Since its smooth surface is almost entirely impervious to water, it is neither necessary nor advisable to attempt a waterproofing treatment. The application of shellac has been tried with the result that when the forms were stripped, the shellac adhered to the concrete and peeled the surface from the fibre board. This was removed from the concrete with great difficulty.

When adjoining sheets of fibre board are butted tight water will cause the edges to rise and leave depressions in the concrete. Where design requires joints to suggest stone, this characteristic can be used to advantage by cutting the fibre board to the required stone sizes. If no joints are desired, such depressions can be prevented by allowing an expansion gap of not more than $\frac{1}{8}$ in. for full sheets. This gap will leave a small cement fin which can be brushed off easily and with no damage to the surface. Further precaution against surface marring can be taken by burnishing all edges that occur at exterior angles, by driving all nails flush only and by nailing the fibre board to the same form board wherever practicable. In instances where nails have been staggered the action of the form boards has been sufficiently opposite to cause the fibre board to buckle and leave a wavy concrete surface. Depending on the length of time it remains in the forms, fibre board is likely to lose its smooth surface and some of its rigidity, so that the advisability of using it for more than one pour will depend on its condition when released from the forms.



FIG. 2—WESTWOOD BRANCH, SECURITY-FIRST NATIONAL BANK OF LOS ANGELES—MORGAN, WALLS AND CLEMENTS, ARCHITECTS

WOOD FORMS

Except for occasional impressions from raised grain and the absence of fins and joint depressions, the surfaces obtained by the use of wood panels are comparable in texture to those resulting from fibre board. Grain impressions are not usually deep enough to be noticeable except in bright sunlight, but if they are objectionable the grain can be sealed. However, this might be relatively expensive, since the precaution is not necessary in the use of fibre board. When the edges of wood panels are planed smooth and butted tight there is no resultant swelling and it is not necessary to allow an expansion gap between panels. Therefore depressed joints can be obtained in one pouring operation only through the introduction of wood strips cut to required width of the stone joints. The advisability of using the same panels for subsequent pours can again be decided by their condition when stripped from the forms.

All wood milled for form lining should be as smooth as possible, kiln dried and entirely free of loose knots or other defects. Probably the best wood for this purpose is California white pine—generally called sugar pine—and eastern white pine because their uniform grain has a negligible tendency to rise under the action of water. Due to the compactness of the grain, waterproofing is sometimes not necessary for these woods. However,



FIG. 3—DETAIL OF WESTWOOD BRANCH BANK. A RECENT STRUCTURE FOR WHICH FIBRE-BOARD LINED FORMS AND PLASTER MOLDS WERE USED

they are soft and care must be taken not to damage their surfaces in puddling operations.

Paste fillers or shellac and varnish as seals for wood grain are not as satisfactory as lead and oil, because of the difficulty of obtaining a permanent bond that will prevent adhesion to the concrete. Since inferior grades have been known to stain concrete, a high grade of lead and oil should be used and unless the wood is painted on all faces, the water from curing operations will raise the grain by seeping under the lead and oil from the unprotected surfaces. If care is taken in stripping, wood molds can be cleaned and used for subsequent pours according to the repetition of detail.

PLASTER MOLDS

The term waste-molds is a descriptive designation of the plaster molds that are used to obtain ornamented surfaces and that are broken in stripping the forms—wasted so to speak. The amount of damage done to plaster molds in stripping depends on the nature of the ornament, and the extent of undercutting, but in no case can they be removed in a condition suitable for re-use—in fact they sometimes have to be chopped from the concrete. Only in instances where the design is extremely simple is there a possibility of stripping plaster molds without serious damage; and where the design presents such simplicity the molds would doubtless be wood.

From the approved clay models made under the Architects' supervision, plaster reverse molds are made. They are reinforced where necessary, and fitted with wood ties and bracing for building into the forms. In order to prevent the collapse of waste molds under the pressure of the concrete, it is advisable that they be not less than 1 in. thick at any point. Where a number of identical waste molds are required by the repetition of ornament, they are obtained by the use of a glue mold made from the reverse plaster mold. For safety in shipping, plaster molds should be well packed in sawdust.

Unless plaster molds are sized or fixed to prevent the absorption of moisture from the concrete, they may disintegrate and destroy the ornament, or the concrete may soften, spall and leave a rough surface. The application of two coats of thin shellac

has been found the best and cheapest method of sealing plaster molds and while there are several other methods used in an attempt at economy, there is none as simple or efficient as the use of shellac. A mixture of gasoline and melted paraffin which is sometimes used as a substitute for one coat of shellac, seals the mold when the gasoline evaporates and leaves only the paraffin in the pores. One coat of shellac is required after this operation. The difficulty of using the gasoline mixture lies in the fact that the gasoline must be luke warm in order not to congeal the melted



FIG. 4—PLASTER WASTE MOLD SET IN PLACE

paraffin—and heating gasoline is not the safest of operations. There has also been some experimentation with slightly colored lacquer as a substitute for the initial coat of shellac. However, since the best results have been obtained with shellac and inasmuch as one coat of thin shellac has always been required to complete the waterproofing, it seems advisable to use shellac as the sole fixing agent.

Wherever pointing or patching of any kind is necessary for any type of form the best agent is a mixture of plaster of paris and shellac. This mixture sets quickly and leave a hard, tough

surface that can be sand papered, filed or polished, according to the nature of the surface desired.

OILING FORMS AND MOLDS

After the lining materials have been installed in the form work, they must be oiled to simplify stripping. Wood



FIG. 5—FORMS PARTIALLY LINED WITH FIBRE BOARD

molds that are not painted are given a coat of standard liquid form oil which soaks into the wood and requires no further attention. All lining throughout is next greased with a form oil that has a consistency of cup grease and finally wiped as clean as possible with rags, for any excess oil will cause unsightly stains on the finished surface.

Numerous methods are used with uniform success in greasing waste molds and which employ all kinds of lubricants from cylinder oil to cup grease. In one case cup grease is thinned to the point where it can be applied with a brush, by the addition of a mixture of crystallized stearic acid and a small quantity of kerosene.

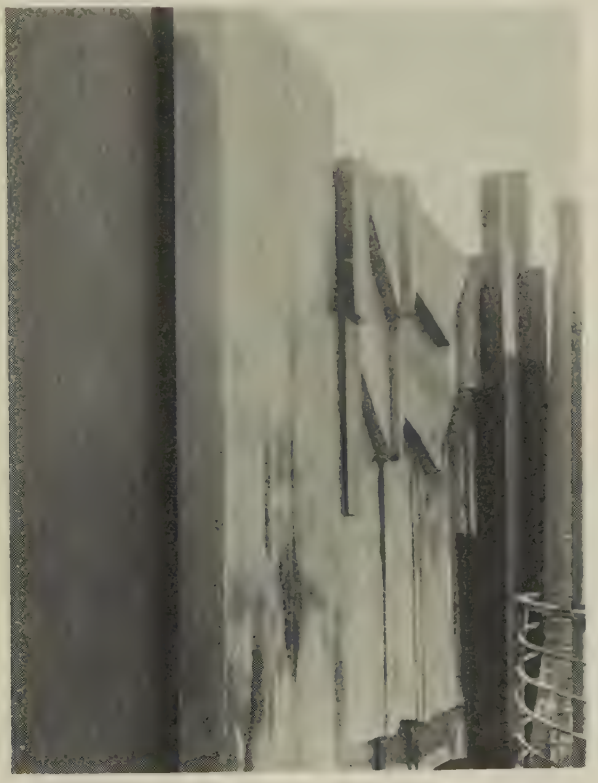


FIG. 6—WASTE MOLDS AND FIBRE BOARD IN PLACE

For anchoring the two sides of the form work, pencil wire ties equipped with collars and set screws are preferred to bolts, because they can be removed with less damage to the concrete. The tendency of such wire to stretch under pressure can be overcome by proper spacing of the ties according to conditions.

CONCRETE MIXTURES

The study of the various concrete ingredients and their proportions in the mixture is the most important step in developing satisfactory exposed concrete surfaces, because it controls the workability and the limits of adaptability of the concrete to

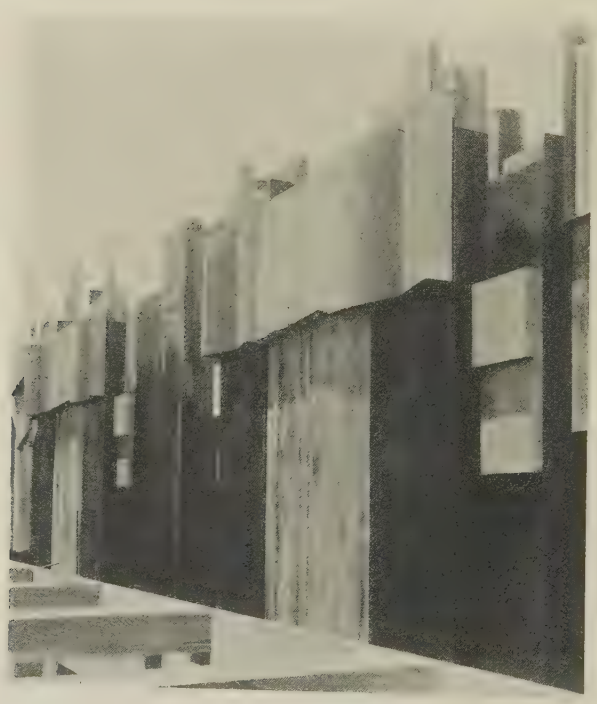


FIG. 7—WASTE MOLDS, FIBRE BOARD AND WOOD MOLDS IN PLACE

conditions of forms and ornament. Concrete control embraces the selection of aggregates, cement, sand, admixtures and water, and the testing of these materials in combination and for impurities.

The proportions of ingredients in the concrete mixture will be established by local building ordinances and by the water-cement ratio established for the developed strength required in the concrete. It seems a universal tendency to place concrete too wet and since excessive water is ruinous to concrete surfaces,



FIG. 8—WASTE MOLD IN PLACE

great care must be taken to prevent a surplus of water. The success of the concrete finish is dependent upon the proper suspension of the materials. A minimum of water helps to prevent the weakening of concrete and the formation of rock pockets and voids. There are conditions of difficult placing such as undercutting in waste molds or projections beyond the normal wall faces that demand a mixture of greater water content for proper workability, but this extra water is offset by a corresponding increase in cement. In all such cases of enriched mixtures, slump tests will determine the limits of richness. For plain surfaces proper workability requires a slump of approximately 4¾ in. in a specimen cone of a 6-in. diameter and 12-in. high. For varying conditions a mixture with as much as a 7-in. to 8-in. slump is often necessary. Similarly the concrete may vary from a mixture of 1 cement to 2½ sand, to 3½ rock or gravel, to a mixture of 1:3:3 or richer. For acceptance the concrete must develop a compressive strength of not less than 2000 lbs. per sq. in. in 28 days. Conditional acceptance at a seven day period

requires not less than 900 lbs. per sq. in. A survey of a series of tests shows that for a slump of 7-in. to 8-in. the concrete has developed compressive strengths that vary from 1000 to 1800 lb. per sq. in. after 7 days.

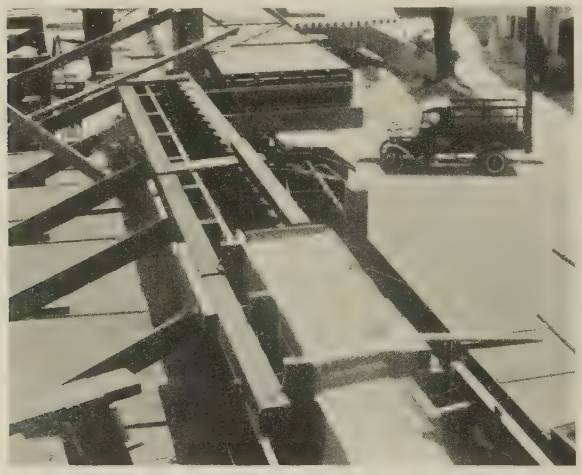


FIG. 9—CONCRETE PLACED AGAINST WOOD MOLDS

ADMIXTURES

Numerous properties are claimed for various concrete admixtures, among which are the tendencies to increase the workability of the mixture and to prevent efflorescence. From the standpoint of workability and finish alone, we have recently obtained more satisfactory surfaces by substituting an equivalent amount of extra cement for the admixture previously used. However, we do not necessarily subscribe to the idea that the use of admixtures should be abandoned for there is no doubt that they tend to increase the strength and durability of concrete, and admixtures which change the structure of hydrated lime are effective preventives of efflorescence. Other steps to prevent efflorescence are the usual rigid specifications governing the properties of the cement, sand, aggregate and water.



FIG. 10—TYPICAL ARCHITECTURAL CONCRETE SURFACE SHOWING WIRE AND MINUTE AIR HOLES AND DEPRESSION FROM A FIBRE BOARD JOINT

PLACING CONCRETE

Before concrete is deposited all debris and drippings should be removed and the forms should be thoroughly wetted. To insure a uniformity of surface texture, the concrete should be transported as rapidly as practicable and as near as possible to its final position to avoid rehandling and so placed in the forms that no separation will occur. It should be deposited in continuous horizontal layers with a difference in levels not greater than 18 in., in order that there be a minimum flow of the concrete. Throughout the placing operations the concrete should be assisted into place by constant puddling to insure a uniform filling of all voids and an elimination of rock and air pockets by forcing air and excess water to rise to the top of the pour. By tapping the form boards the sand and cement are drawn to the

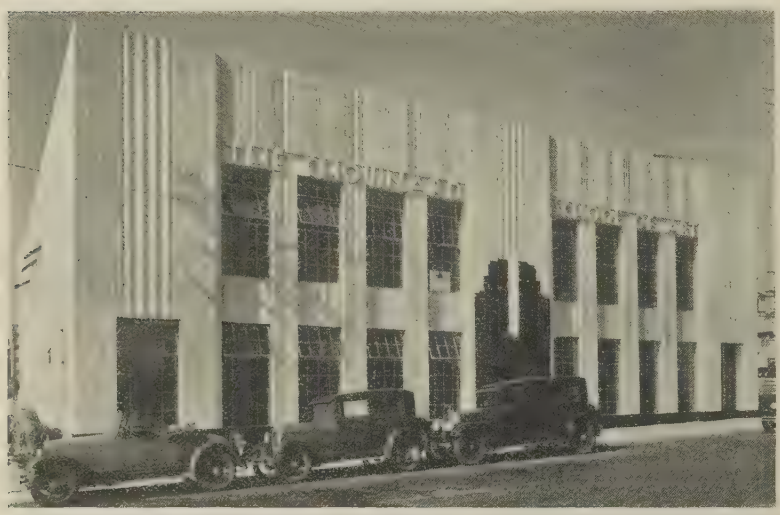


FIG. 11—CHOUINARD SCHOOL OF ART, LOS ANGELES—MORGAN, WALLS AND CLEMENTS, ARCHITECTS

outside of the wall and produce a smooth surface free from exposed aggregate. For agitating the concrete during the pour, puddling with wood strips is a safer operation than spading, because there is less opportunity to cut wire ties or to mar the surface of the form lining.

When it has been determined where the pour is to stop a wood strip is tacked to the forms at the outside surface of the concrete wall. The pour is carried about ½ in. above the underside of the strip and sloped to the inside face so that laitance and excess water will drain to the inside face of the concrete which will be cleaned and plastered. About an hour later the strip should be removed and the concrete leveled off with a steel trowel so that any demarcation between the subsequent pour will be a straight line and therefore least noticeable.

STRIPPING FORMS

Forms should ordinarily remain five to seven days before stripping and during this time the form work for the next pour is constructed. Before the next higher pour is made, the bottom



FIG. 12—CONCRETE WAS PLACED AGAINST MILLED WOOD FORMS AND SURFACES THROUGHOUT BRUSHED WITH CEMENT WASH AS FINISHING MATERIAL



FIG. 13—DETAIL CHOUINARD SCHOOL OF ART

whaler of the new form work is pulled as tight as possible against the top of the concrete previously poured to prevent soiling this surface by leakage of water and cement. Whatever water or laitance leaks through the bottom of the forms must be washed from the lower surface immediately in order to prevent permanent damage.

The removal of forms is an operation that requires extraordinary care to prevent damage to the concrete surface, and it is in this operation that the limitations of design become apparent. Design should provide means of disguising the variations between pours either by actual or painted architectural joints or by applied surface treatments. A mixture of cement and water applied with a brush will conceal slight discolorations and retain the natural color of the concrete.

For ease of stripping there should be no exterior acute angles. To allow for the uneven expansion of forms all exterior angles should be slightly obtuse wherever practicable because of the difficulty of removing the forms without spalling. On wide pilasters where a number of exterior angles occur, a series of kerfs to within ½ in. of the inside face and about 3 in. apart is a very effective method of relieving the possibility of damage to corners by absorbing the expansion of wood molds.

PATCHING AND CURING

To prevent extreme variations in color, it is important that the proportions of the concrete remain constant for similar and contiguous conditions and that the same cement mortar mixture used in the concrete be employed in patching spalls, voids and wire holes. Patching should be done immediately after the surfaces have been inspected and cleaned of all laitance and projecting fins.

Satisfactory curing requires that the concrete be kept wet for at least seven days, but this period varies according to the structural conditions of the concrete. After proper curing the surfaces should be brushed clean of all excess cement that might roll up under a painter's brush or tend to defeat any efforts to apply a finishing material.

Readers are referred to the JOURNAL *for March, 1931, for discussion which may develop. Such discussion should reach the Secretary by February 1, 1931.*

Design and Construction of Bonnet Carre Spillway

*With Special Reference to Concrete**

BY HELMER SWENHOLT†

INTRODUCTION

The bonnet carre spillway now under construction on the left bank of the Mississippi river, about 30 miles above New Orleans, is a controlled spillway 7700 ft. long, emptying in time of unusual flood through a leveed floodway into Lake Pontchartrain, six miles away. Authorized under the Flood Control Act of May 15, 1928, according to the adopted plan it is to be capable of discharging 250,000 sec. ft., leaving 1,250,000 sec. ft. to go by New Orleans, preventing the stage at Carrollton from rising above 20 ft. on the gage. This plan is the outcome of 200 years of discussion and efforts to control the flood waters of the Mississippi river by levees only.

HISTORY

The history of the lower Mississippi River is one of floods. Early settlers along the stream began the protection of their holdings from river encroachment as soon as residence was established. These holdings, in general, were in large grants, and plantations became the center of social and business activity. Small levees were built along their fronts, joining with neighboring levees, or extending rearward towards the swamps for flank protection. A levee along the New Orleans front was completed as early as 1727. Levee building by State and local interests was stimulated by the swamp land act of 1850 following the damage of the floods of 1849 and 1850, and levees were extended and built higher and higher as the South Central States advanced

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industrially and agriculturally. Floods kept pace with levee heights, frequently defeating the builders, the river spreading its surplus waters over the adjacent countryside through crevasses and finding its way to the Gulf, or perhaps back to its own channel through bayous, adjoining rivers and lakes. The confinement policy of flood control terminated with the disastrous flood of 1927, and as a direct result of this flood the Flood Control Act of 1928 was passed and the Bonnet Carre Spillway, a subfeature of the adopted plan, was authorized.

This spillway was discussed as early as 1816 and much interesting and instructive data exist in studies made at various times, including the investigation of the Corps of Topographical Engineers in 1850, the Humphreys and Abbott report of 1861, and others. As the result of the flood of 1922, Congress in 1926 authorized a Spillway Board, and its recommendations in House Document No. 95, 70th Congress are the basis for the present design.

DESCRIPTION

The plan adopted by the Flood Control Act of May 15, 1928, provides for the safe discharge of 1,500,000 cu. ft. per sec. below Old River between the main river levees, in detail as follows:

114. At Bonnet Carre a controlled spillway emptying into a leveed floodway to Lake Pontchartrain is to be constructed capable of discharging 250,000 sec. ft. This leaves 1,250,000 sec. ft. to go by New Orleans, and should prevent the stage at Carrollton from rising above 20 on the gauge.

115. The Bonnet Carre spillway is so designed as to afford complete control of the discharge into Lake Pontchartrain. This discharge will be begun when the flood stage at New Orleans has reached 20 on the Carrollton gage; it will be regulated to prevent the stage rising above 20, and will cut off as soon as the stage has fallen below that figure. Past records indicate that its operation will be required about once in five years; and for a period of from one to three months during each flood. The discharge through the spillway, in a maximum flood, will be about 10 per cent of the total flow at that latitude. Some of the silt carried by the discharge will be deposited in the floodway itself, the bulk of it in the upper end of Lake Pontchartrain. With the infrequent and limited periods of operation, the silting would not exceed an average of ¼ in. of depth per annum, even if all the silt content were deposited in the lake within 10 miles of the floodway outlet. The average rate of silting for the entire lake would be $\frac{1}{32}$ in. per annum. Discolored water from the discharge may, in a high flood, extend well into Mississippi Sound, but there will be no appreciable silt content remaining in the water at such a distance. The spillway is especially heavily designed to prevent even the remote possi-

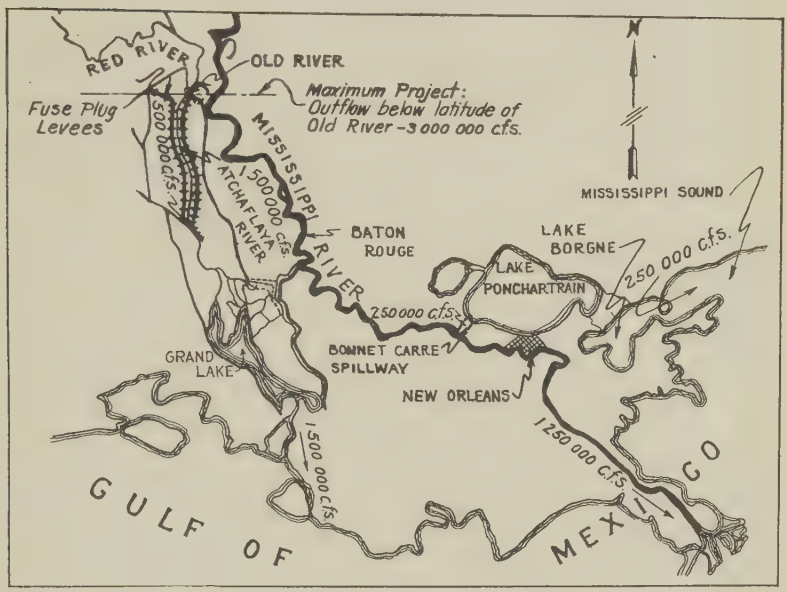


FIG. 1—MAP SHOWING ESTIMATED QUANTITIES OF DISCHARGE THROUGH VARIOUS CHANNELS

bility that the discharge will cut a channel from the river into Lake Pontchartrain.

116. The City of New Orleans has provided, with its own funds, levees on its river front of so wide a crown, and with such flat slopes, as to afford an abundant margin of safety against failure except by overtopping or by bank caving. After the Bonnet Carre Spillway has been constructed, there can be no fear that the flood stages will approach dangerously close to the levee top. The danger of bank caving is minimized by the revetment which has been placed on the under-water bank. The Bonnet Carre spillway will reduce the high current velocities now occurring in great floods. Further insurance can be provided by widening the levees to introduce any desired factor of safety.

In accordance with this plan a site was selected after detailed topographic surveys covering a large area in the vicinity of Bonnet Carre Point, 30 miles above the City of New Orleans. As in other parts of the Alluvial Valley, the river here flows between banks considerably higher than lands back from the river, the banks being about 14 ft. above mean Gulf level, sloping to Gulf level and even lower towards Lake Pontchartrain.

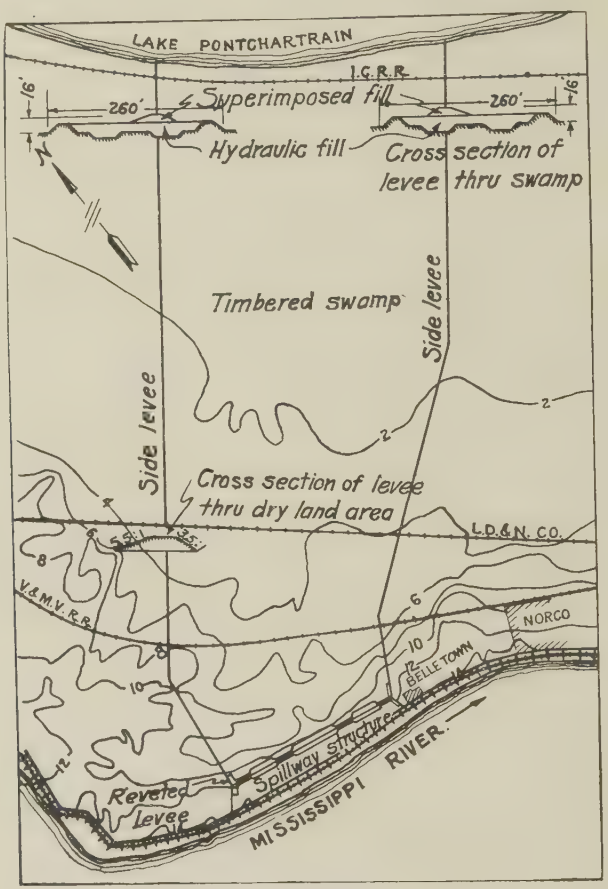


FIG. 2—LOCATION MAP BONNET CARRE SPILLWAY

Of the 7800 acres required, 30 per cent is cultivatable land from 4 to 14 ft. above mean Gulf level, while the remaining 70 per cent bordering on the lake is a low lying timbered swamp, much of the soil consisting of humus and muck. The floodway is crossed by three railroads, one highway, telephone, telegraph and power transmission lines, and a natural gas pipe line. The selected site is shown in Figs. 1 and 2.

DESIGN

In the design of the spillway the following principles governed:

(a) The discharge capacity of the weir must be at least equal to the required discharge under working conditions.

(b) The floodway capacity must be similarly ample.

(c) The structure must be safe, workable and subject to control.

Since the project involved the expenditure of many millions of dollars, it was felt that studies and experiments should be undertaken with great thoroughness. The economic desirability of the project having been previously determined these studies were largely technical, the economic feature entering only into details of design. These technical studies were unusual in that the structure is to rest on an alluvial soil and its operation is to cover all conditions of flow and backwater from no flow to full capacity.

A hydraulic laboratory was established on the site where various scale models were tested for performance. These tests were conducted to determine:

(a) The design which would most effectively dissipate the energy of falling water under any condition of flow.

(b) A design which would withstand such flow and energy as were not dissipated.

(c) A design which would cause the higher velocities of the water leaving the apron to be on the surface instead of at the bottom.

(d) The discharge capacity of weir types.

(e) The velocities at which scouring would occur.

(f) The velocities at which silting would occur.

(g) The rip-rap and toe protection required.

(h) Back water conditions and flow in the floodway.

Foundation tests were conducted also with a view to determining:

(a) The physical characteristics of the soil.

(b) The permeability of the soil.

(c) The bearing capacity of the soil.

(d) The type of foundation most suitable.

These tests and experiments were continued over several months, and influenced features of the design shown in Fig. 3.

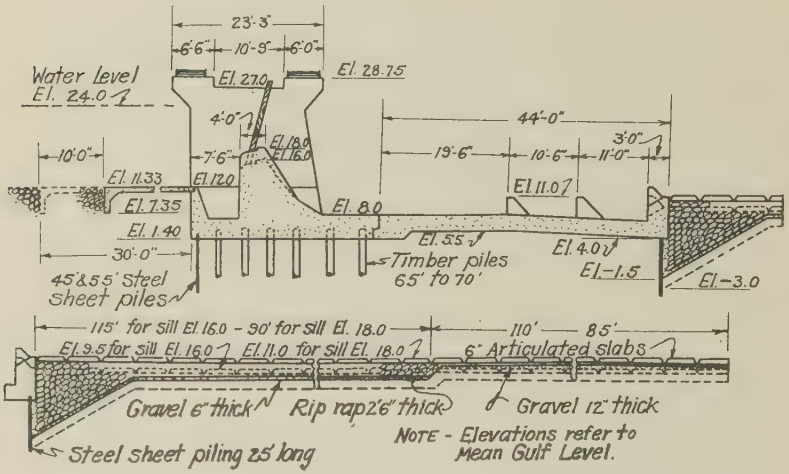


FIG. 3—FINAL DESIGN OF BONNET CARRE SPILLWAY STRUCTURE

The discharge capacity of the structure, which must be at least 250,000 cu. ft. of water per sec., is affected by several factors. It is reduced as backwater in the floodway is built up and the weir becomes submerged; it is increased by the head from a rising river surface above the weir. Computations gave the probable backwater height at the weir; gage relationships between Carrollton at 20 and Bonnet Carre gave a river surface elevation at Bonnet Carre of approximately 24. Discharge capacity was then determined. The concrete structure, 7700 ft. long with 350 20-ft. bays has the required capacity with an ample margin of safety.

FOUNDATIONS

The spillway structure proper rests on untreated wood piles 65 to 70 ft. long. Untreated piles were selected as most economical since their life when cut off below the ground water line is unlimited. The length of these piles conforms to New Orleans practice and the loads are transferred to a safe depth. No point bearing was secured during tests but loads of as much as 90 tons caused no failure of skin friction. The piles are spaced to meet conditions of eccentric loading, the average being 14 tons per

pile, which is standard locally. The fore apron 2.5 ft. thick is separated from the main structure by an expansion joint and rests upon the ground. Experiments indicated that the natural soil should carry this load without undue settlement.

CUT OFF WALLS

A cut off wall of interlocking steel sheet piling 45 to 55 ft. long, driven under the upstream edge of the structure, lengthens the line of percolation. The longer piling was driven to protect a stretch of softer material 1200 ft. along the site. Piling of an arch web section, with a flange and web thickness not less than $\frac{3}{8}$ in., an interlocking strength not less than 7000 lb. per lin. in. of pile, and a section modulus of 3.2 in. cubed was used. It has a copper bearing content of 0.2 per cent. A down-stream curtain wall of similar steel piling 25 ft. long was driven, at the foot of the stilling basin. This wall not only lengthens the percolation line but protects against scour which may occur below the structure. Cut off walls of 25 ft. steel piling are driven between up and down stream piling at intervals of 460 ft.

ABUTMENTS

The abutments at each end of the structure are gravity sections set on piles. A core wall resting on steel sheet piling extends fifty feet into each side levee, and wing walls, making an angle of 15 degrees with the central section, extend 100 ft. each way from these sections. These abutments were built first so that levees behind them would have ample time for settlement before being revetted. This revetment consists of a continuous concrete slab 4 in. thick, wire mesh reinforced, protected with a short cut off wall and rip-rapped.

WEIR

The weir is designed as a gravity section, sufficiently heavy to withstand lateral and uplift pressures. Its face is so shaped as to induce an abrupt drop of water, the eddy under the nappe tending to dissipate more effectively the energy of the falling water. A 4-in air vent through the piers will prevent the formation of a vacuum under this lower nappe. Sill castings to meet the operating needs of the needle type of dam are imbedded in the concrete at the crest of the weir. For maximum efficiency



FIG. 4—POURING FORE APRON

and convenience of operation these sills are arranged in alternating groups. Commencing at each abutment, 44 bays have sills at elevation 18, the next 44 consecutive bays having the sills at elevation 16, etc. This arrangement will be seen to provide for a center section consisting of 88 bays having sills 16 ft. above mean Gulf level. Expansion joints ½ in. wide and filled with an approved asphalt filler are provided at the center of every other weir section. These joints extend across the fore apron and stilling basin which lies below the structure proper, and are sealed on the upper side by a 12-in. copper strip extending from the steel sheet piling to the weir crest.

PIERS

The piers are 2 ft. thick, spaced 22 ft. center to center, reinforced with ½ in. bars set vertically. They are designed to withstand safely the pressure of water flowing through an opening on one side only. They are poured in monolith with a half section of weir on each side. The functions of the piers are three fold; they transmit a proportion of the water pressure received through the needles to the foundation; they divide the spillway into compartments whereby flow may be controlled; they carry service bridges from which the spillway is operated and escape bars which support the upper ends of the needles.

NEEDLES

After a study of various types of gates the needle method for control of flow over the spillway was adopted in the interests of economy, simplicity and certainty of operation. The needles are of 8 in. x 12 in. timbers resting on a sill at the weir crest, their tops supported by horizontal escape bars suspended between piers. Spaces between them may be closed by battens to prevent excessive leakage. A full bay may be released simultaneously by tripping the escape bar, but ordinary operation provides for lifting individual needles by means of hand cranes on the service bridges which surmount the piers. These needles when raised will be stored by hooking the lifted ends to one of the service bridges the other ends resting on the escape bar. The service bridges are standard gauge tracks, the stringers of each consisting of two 15 in. 33.9 lb. channels, to form rails for hand cars, the cross bracing forming the beams upon which the decking rests.

FORE APRON AND STILLING BASIN

The fore apron and stilling basin as shown in Fig. 3 consists of a concrete slab, 2.5 ft. in thickness, not reinforced. It is underslung in anticipation of possible settlement and as a better seal against upward pressure, as well as against currents flowing over the weir and apron. It is connected by re-inforcing bars to the main structure. The joint is treated as an expansion joint and has an asphalt filler. The design of this fore apron and stilling basin is regarded as one of the outstanding results of experiments made with the model spillway. The width of apron, location and shape of baffles, height of secondary weir and depth of stilling basin were all worked out in small scale tests which had two objectives: (a) to convert high spillway velocities to mean floodway velocities, and (b) to reduce the bottom velocity of water leaving the stilling basin to 50 per cent of the surface velocities.

A rear apron 9 in. thick, mesh reinforced, and 30 ft. wide lies above the weir. Its purpose is to prevent scour and it serves also to lengthen the percolation line under the dam. There is a shallow cut-off wall on the up-stream edge and a protection of rip rap.

TALUS

The talus of rather unusual design was worked out in model tests. Excavation of varying depth is carried about 200 ft. wide below the baffle wall and on this is laid a 6-in. surface of 3 to 6-in. spalls. Rip rap stone is next placed to a depth of 14 ft. at the wall and 2½ ft., 16 ft. beyond, this latter depth extending to 100 ft., the remaining distance being covered with gravel or spalls only. Over this entire area is poured a flexible concrete mat of considerable extensibility. The mat units are 3 ft. x 3 ft. x 6 in., fastened together, slackly, with copper bearing steel wire which is continued through the slabs as reinforcing. The slabs are beveled on the upper side, better to deflect the water upward, and the whole mat is fastened to the toe wall of the structure by means of reinforcing rods looped through Everdur bars, allowing 2 ft. of settlement at this point. Holes are left in each block for emergency ties.

FLOODWAY AND SIDE LEVEES

The discharge of the spillway will be carried to Lake Pontchartrain through a floodway of 7,800 acres, between levees 7,700 ft. apart at the structure increasing to 12,000 ft. at the lake. The upper levee has a length of 6.27 miles and the lower 5.52 miles. Approximately two miles of each levee back from the river is of the ordinary dry land type of construction; the balance extending to Lake Pontchartrain is being built by hydraulic methods to the section shown in Fig. 2.

As mentioned previously the floodway must be ample to pass the 250,000 sec. ft. discharged over the spillway. A thorough study was made, not only with model tests, but of experiments and records of other channels and floodways. Backwater curves were calculated using the formulas of Chezy and Kutter. Especial weight was given to the Department of Agriculture's determination of the roughness factor in the St. Francis Floodway. A final curve was worked out whereby an economic width is secured, levee heights are not excessive, velocities will not induce scour, and weir submergence will not exceed 60 per cent which is not objectionable. These conditions will be obtained when the floodway is cleared of undergrowth and small trees, leaving stubble and stumps not more than one foot high, and allowing trees of greater diameter than one foot to stand.

The levees are being built with ample freeboard to care for the maximum flow with an added factor of safety to provide for possible floodway deterioration from silting. The levees on the high alluvial land back from the river are of 17 ft. average height, with a crown width 12 ft. and slopes of 5.5 to 1 land side and 3.5 to 1 floodway side. The levees through the swamp section where the top soil is largely of muck and humus require a different construction. A base width of 260 ft. is cleared and two 40-ft. muck ditches dug, the material from which is used to build small confining levees. The space between these levees is then filled hydraulically to a height of about 10 ft. and after this base has dried out there will be built a superimposed levee of standard section to the required grade. About 8,000,000 cu. yd. of material will be placed in these levees. They are under contract and, except for the superimposed sections, it is expected that they will be completed early in 1931.

CONSTRUCTION OF SPILLWAY STRUCTURE

The contract for the spillway structure proper was let in August, 1929, and work started almost immediately. The quantities involved, the unit prices, and the status of the work on June 30, 1930, are shown in the following table:

Items	Estimated Quantities	Unit Price	Total Quantity to Date	Per cent Complete to Date
				June 30, 1930
Common excavation	420,000 cu. yd.	.40	332,296.	79.12
Back fill	8,100 cu. yd.	.37½	2,623.9	32.39
Levee fill	75,000 cu. yd.	.20	61,250	81.67
Steel sheet piling	600,000 sq. ft.	.80	583,104.69	100.00
Steel sheet piling, cut-off	———	———	91.	———
Wood piling	750,000 lin. ft.	.43	746,279.5	100.00
Wood piling, cut-off	———	———	11,076.	———
Class "A" concrete, structural	104,000 cu. yd.	9.50	78,581.4	75.56
Class "B" concrete, revetment	23,000 cu. yd.	10.00	693.	3.01
Rip rap	128,000 tons	———	22,612.95	17.67
Gravel (or spalls) except for concrete	70,000 tons	3.75	21,251.08	30.36
Castings	532,000 lb.	.06½	361,634.97	67.98
Bolts, etc.	75,000 lb.	.08¾	48,047.68	64.06
Reinforcing steel	280,000 lb.	.04½	253,359.19	90.48
Ladder rods	6,200 lb.	.44	3,627	58.50
Structural steel	500,000 lb.	.056	8,018.24	1.60
Reinforcing mesh	122,000 lb.	.07	1,263	1.04
Copper bearing steel reinforcement	537,000 lb.	.047	17,732.55	3.30

This contract awarded to one firm, involves $3,065,957; the service bridges and needles, let separately, will cost an additional $193,933. Completion is required in 530 days after notice to proceed and present progress indicates that the contractor will be well within the limit set.

Excavation, well ahead of pile driving, was done with a 1¼-yd. 50-ft. boom dragline, and the material hauled in dump cars to the side levees over standard gage work track with gasoline locomotives. Excavation was made to grade, but the driving of piles and some inflow of material made much hand work necessary.

Wood piling was driven with two drivers, and steel sheet piling with two smaller drivers and one caterpillar crane with swinging leads, this latter used on the short piling only. Driving was kept not less than 400 ft. in advance of concrete pouring to prevent vibration as well as other interference with that work. With the exception of one short stretch of the steel sheet piling, no particular difficulty was experienced in driving either wood or steel piling and this phase of the contract was finished ahead of schedule, and without interference or delay to the pouring of the weir structure, which began as soon as the abutments had taken a proper set.

CONCRETE

Concrete is the largest single item in the cost of the spillway structure and through sheer bulk and length of distribution is of considerable interest to engineers. One hundred thirty-three thousand cu. yd. is being poured over a stretch of 7,700 ft. and many problems have arisen in its proportioning and handling. Class "A", structural, 110,000 cu. yd., includes the weir and pier structure, aprons, abutment and core walls, and must have a water-cement ratio of .87 (6½ gal. of water per sack of cement) and a minimum compressive strength of 2,250 lb. per sq. in. at 28 days. Class "B," revetment, 23,000 cu. yd., is used in the flexible slab talus and the revetment for side levees, and must have a water-cement ratio of 1.07 (7½ gal. of water per sack of cement) and a minimum compressive strength of 1500 lb. per sq. in. at the same age. Durability and impermeability are the objects of the design; the high strength required is necessarily developed in the mixtures specified.

TESTING AND INSPECTION

A laboratory was established at the spillway site before construction was started, and complete preliminary investigations were made to determine the mix to be used. Studies were made of the fitness of available materials, the cement factor, the most desirable quality of concrete and the approximate cost.

Concrete aggregates, secured from three pits on the Tangipahoa river in Southeastern Louisiana were clean and hard, with negative colorimetric tests, containing about 0.2 per cent clay and silt, and having the following sieve analyses:



FIG. 5 DRIVING STEEL SHEET PILING — UPPER CURTAIN

GRAVEL

Size	Pit No. 1	Pit No. 2	Pit No. 3
1½ in.	0	0	0
¾ in.	28.2	26.6	25.9
⅜ in.	58.5	58.7	61.2
No. 4 sieve	6.7	8.1	8.0
No. 8 sieve	6.6	6.6	4.9
F. M.	7.08	7.05	7.07
Unit wt. lb.	101.	101.	100.
Spec. Gravity	2.64	2.65	2.64

SAND

Sieve No.	Pit No. 1	Pit No. 2	Pit No. 3
4	.4	.4	1.7
8	9.3	16.5	9.6
14	8.7	14.6	7.3
28	18.8	21.6	18.5
48	45.1	34.2	50.1
100	17.7	12.7	12.8
F. M.	2.48	2.89	2.56
Unit Wt. lb.	110.	113.	108.
Spec. Gravity	2.63	2.66	2.62

These tables would indicate that the aggregates were similar in gradation, but variation in shipment from any of the three pits is greater than the average difference between them. It was felt, therefore, that daily sieve analyses would be necessary to keep the mix workable and the concrete uniform, and to avoid too great fluctuation of the cement factor. The cement used comes from Alabama in bulk, is almost wholly free from lumps, and is shown by 7- and 28-day briquette tests to have a strength 25 per cent in excess of the A. S. T. M. standards. Ordinary Mississippi river water is used, and while it has a high silt content, no objectionable results have been obtained; in fact some of the high strength obtained in tests may be due to this presence of silt in the water. Preliminary investigations showed that materials were excellent, though poorly graded, and that good concrete could be made from them. As anticipated, however, actual field conditions and differences in the aggregate have made necessary constant revision in the mix as designed. The cement

factor has varied from 5.65 to 5.91 sacks of cement per cu. yd. of concrete, but the cost of raw materials has at no time exceeded $4.28 per yard.

The concrete inspection and field testing laboratory is under one head, thus unifying what are often two distinct departments, and making possible an immediate and constant check on all phases of the work and close co-ordination of action. An outside laboratory makes mill tests of all cement purchased, furnishing copies of results to the field laboratory. Cars are sampled and tested as received, and bin tests are made to prevent use of damaged or deteriorated cement. Aggregates are tested as received and before unloading, and samples sent to the laboratory for checking and recording. At least twice daily, cylinders are made from the concrete at the mixer and again after it has been placed in the forms. The mixer samples are given laboratory curing, those from the forms are cured under field conditions; careful records of both types are kept and parallel graphs of compressive strength at 7 days, 28 days, 3 months and 6 months, are plotted. Weight is given to precipitation, laboratory and ambient temperatures and relative humidity, and a composite chart of local weather conditions is plotted from observations taken three times daily.

MIXING

A mixing plant, central with reference to the longitudinal axis of the weir serves the purpose of the work. Here are installed two 1-yd. mixers equipped with inundators. Cement is lifted to the storage bins by compressed air and fed to the hoppers through a weighing device which closely checks the actual cement consumption and offers a practical gage for the theoretical cement factor. A 3-in. slump is desirable from the standpoint of workability, but is maintained with difficulty, due apparently to variation in the size of aggregates. It is necessary also to keep a constant check on the moisture content of the aggregate, particularly after a rain, and compensate for extra moisture with additional cement to maintain the proper ratio. Mixing is continued at least 1½ min., with 12 revolutions of the drum traveling at a rate of 200 ft. per min. The finished concrete is then dropped into buckets, which are conveyed by truck to the forms for pouring.

Transportation during the winter months and prior to construction of the fore apron, was hindered by soft roads, necessitating the use of mats. This difficulty was overcome, however, and for several months it has been possible to operate at the full capacity of 56 yd. per hr., a fleet of 4½ ton trucks, each carrying two 1-yd. buckets, hauling the concrete to the site of the work where it is lifted and poured by a caterpillar crane.

CHARACTER AND ORDER OF CONCRETE CONSTRUCTION

The specifications for concrete work require puddling along the face of all forms, the surface to be free from voids, and no plastering permitted. Defective concrete must be removed and new concrete keyed, dovetailed or anchored in its place. All top surfaces have a float finish without mortar, and all angles are rounded or beveled unless definitely specified otherwise. A curing period of 12 days is required for all horizontal surfaces, and 10 days for vertical surfaces. Water curing is used though wide latitude is allowed.

The two abutments were constructed first to give ample time for placement and settlement of adjoining levee before revetting with concrete. Forms of surfaced lumber, sealed with paraffin oil, were used; the sub-foundation was roughly surfaced and spread with a coating of sand, and the wood and steel sheet piling were scrubbed clean. Particular care was used in the puddling and placing of concrete, the result being walls entirely free from honeycombing. Some difficulty was experienced on the first abutment in holding the forms true to line, caused by the high lifts attempted, 13 to 15 ft. This height was reduced by half on the second abutment and no such trouble occurred.

Construction of the fore apron and baffle seats was started as soon as the abutments were finished. Beginning at the west end, alternate sections 22 ft. long, were poured. The two side bulkheads and rear face forms were of wood and the forestepped bulkhead was of steel. Side bulkheads were omitted when the intermediate slabs were poured. Perfect alignment of the baffle seat insets proved difficult until a special bridge was constructed and the sections lined separately with a transit.



FIG. 6—PLACING FORMS FOR WEIR SECTION



FIG. 7—CUT-OFF DITCHES—SWAMP SECTION LEVEE BASE

Pouring of the weir and pier base was started in December, 1929. This is a heavy slab, 4 ft. deep, built in 22 ft. sections with expansion joints at 44 ft. intervals. Upon this base rests the pier weir section started in January, 1930. Steel forms are used. They are fitted together to form a mold for a pier and half section of weir on each side. Pouring is continuous to the weir crest where a construction joint is prepared and the upper part is continuous to the weir crest where a construction joint is prepared and the upper part of the pier poured separately.

CONTRACTS

The contract for the entire spillway structure except for service bridges and needles is being undertaken by Stevens Brothers, St. Paul, Minn., and The Miller Hutchinson Co., Ocean Springs, Miss., combined. Doullut and Ewin, Inc., New Orleans, who will erect the service bridges and needles, also drove all piling under sub-contract. The dry land levees are being built by Boone and Wester, Sellers, La., the lower hydraulic section by the McWilliams Dredging Co., New Orleans, and the upper hydraulic section by Jahncke Service, Inc., New Orleans.

CONVENTION DISCUSSION

B. Moreell (Washington, D. C.): I would like to ask Captain Swenholt about the reinforcement for the flexible mat. I understood that copper bearing steel wire was to be used. Is that right, or is it copper covered?

Captain Swenholt: Copper bearing steel wire was decided on. There was a great deal of difference in the first cost. The copper bearing steel was believed amply good, and in case of deterioration, we could fasten the slabs together again, so rather than go to that additional expense, we selected the copper bearing wire. As a matter of fact, the real reason was the difference in first cost.

B. Moreell: How was the bearing power of the wood piles determined? Were they driven by formula?

Captain Swenholt: The bearing power of the wood piles was determined as the result of several things. In the first place, we conducted tests at the spillway to get the bearing power. We then employed J. F. Coleman, New Orleans, as a consulting engineer, and he, in conjunction with others, studied the results of these tests. The recommendation of Mr. Coleman was fourteen tons for each pile, and that was adopted.

B. Moreell: I think you do not understand my question. I want to know if you pre-determined the length of the pile and then drove that length, or determined the bearing power of the pile as you drove it, from the *Engineering News* formula?

Captain Swenholt: We tested the piles after they were driven. We computed the bearing capacity of piles from the *Engineering News* formula, but it did not seem to check with the actual results we got on our tests at the structure.

B. Moreell: In other words, you made a load test and then determined the number of piles required from your load test?

Captain Swenholt: Yes, sir.

Readers are referred to the JOURNAL *for March 1931, for discussion which may develop. Such discussion should reach the Secretary by February, 1, 1931.*

Composite Columns

BY L. J. MENSCH*

The composite column here considered is of hooped concrete with standard spiral and vertical reinforcement and with an additional heavy strengthening member or members of cast iron or steel inside the hooping. The spiral reinforcing is generally 1 to 1½ per cent, the vertical steel bars 1 to 4 per cent and the heavy metal core from 2 to 50 per cent of the area inside the hooping. Standard hooped concrete columns are considerably larger than fireproofed steel columns for loads greater than about 200,000 lb.

In high buildings, loads from one to two million pounds are common, requiring concrete columns from 3 ft. 0 in. to 4 ft. 6 in. square. No wonder architects and owners decide to use steel skeletons, although their cost is at least 6¢ per cubic foot higher than for concrete. In some instances engineers have used steel columns, or even steel skeletons for the lower part of a high building and reinforced concrete for the upper part, but results were not quite satisfactory, in having two different contractors working on the frame, which delayed progress and wasted money. As will be shown, composite columns can be designed to fail at a unit stress of 30,000 lb. per sq. in. of the spiral area, which is a considerably greater load than a fireproofed steel column can carry.

Composite columns were first investigated by the well known concrete pioneer Dr. F. Emperger, remembered in this country as the builder of the first Melan type of arch bridge, 36 years ago. He first tested structural steel columns embedded in plain concrete and found the strength of the combination to be considerably less than the sum of the strengths of the structural steel and

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of the concrete. Cast iron columns embedded in plain concrete gave still more unsatisfactory results. He found that the plain concrete failed first and that the shock of the failure of the concrete made the cast iron fail at a lower load than when tested alone.

The combination of cast iron core with hooped concrete was the next step and Dr. Emperger obtained results which showed that the strength of this combination is greater than the sum of the strength of hooped concrete alone and of the cast iron alone.

The difference in behaviour of plain and hooped concrete columns may be explained by the different compressibility of plain and hooped concrete. Plain concrete fails at a shortening of 0.001 of its length, while hooped concrete fails at a unit deformation of 0.01. At a unit deformation of 0.001, steel is stressed to about 30,000 lb. per sq. in., and cast iron to about 15,000 lb.; At a unit deformation of 0.01, steel is stressed to about 35,000 lb. per sq. in. and cast iron to from 50,000 to 100,000 lb. per sq. in., depending on the grade and provided that the strength is not reduced by deflection, or increased by lateral restraint. Hooped concrete forms such a lateral restraint and the object of all tests made up to this time, has been to find the law governing it. To recognize this restraint we have to know the strength of the metal core without its shell of hooped concrete.

We are still in the dark about the strength of structural steel, as the final report of the special committee of the A. S. C. E. on steel columns and struts clearly showed, and we are still less informed about the strength of cast iron members. For structural steel of heavy sections, and only those are of interest in composite columns, we can assume the ultimate strength for $l/r = 50$, 85, and 150 as 35,000, 32,000 and 23,000 lb. per sq. in., respectively. For cast iron without any defects a formula similar to Tetmayer's formula seems to give the nearest results. The ultimate strength per sq. in. $= 100{,}000 - 1500\ l/r + 7\ (l/r)$.[2]

This formula gives the following values:

l/r	20	30	40	50	60	70	80
Ultimate strength, lb. per sq. in.	72,800	61,300	51,200	42,500	35,200	29,300	22,800

The tests made by Dr. Emperger were under the auspices of the Austrian Committee for Reinforced Concrete and are described in No. 3 and No. 11 (1912 and 1927) of the publications

of that Committee and in No. 11 of the Oester. Wochenschrift fur den Offentlichen Baudienst of 1915.[1] The most important results are summarized in Table 1, while Table 3 shows all the particulars of the highly interesting group of 1926. These latter tests were comprehensive and included six standard hooped concrete columns, with 12 in. spiral of 0.64 to 1.65 per cent, with 1.1 per cent of vertical bars, and of a concrete mixture which varied from 1:8 to 1:4½.



FIG. 1—SECTION OF COMPOSITE COLUMN WITH FOUR VERTICAL STEEL BARS AND FOUR CAST IRON ANGLES. SEE TABLE 3

These columns were tested at ages from 14 to 69 days and gave ultimate strengths of the hooped concrete from 1942 to 5420 lb. per sq. in. Then 14 composite columns were made, having the same dimensions and properties as some of the standard hooped concrete columns, but reinforced in addition by 4 cast iron angles 3.14 x 3.14 in. x .39 in., tied together every 10 in. as shown in Fig. 1. They were tested at the same age as the corresponding standard columns. The difference in strength was ascribed to the cast iron core, and the stresses varied from 60,000 to more than 100,000 lb. per sq. in.

Concrete 14 days old seemed to show as good a stiffening effect as concrete 69 days old and the only positive conclusion we can draw from these tests is that 1 per cent and 1½ per cent spiral reinforcement gave more uniform and slightly higher stiffening effect than smaller spirals. It is true, however, that the l/r of the cast iron core was already very small, namely,

[1]See also Proceedings, American Society of Civil Engineers, 1923, V. 26, p. 1185.

TABLE I—EUROPEAN TESTS

Ratio of Length to Diameter of Spiral	2	4.5	4.9	5.3	6.75	6.8	8	8.7	8.75	9	11.5
Diameter of Column (Inches)	15.25	8	12	16	8	15.25	14	15.25	9.6	8	10.3
Number of Columns Tested	1	1	2	8	1	1	12	1	2	2	3
l/r of Metal Core	16	44	17.5	15	66	57	26.5	72	73	88	79
Per Cent of Metal Core	6	6	9	12	6	6	9	6	7	6	7
Ultimate Stress on Cast Iron, Lb. per Sq. In.	100,000	100,000	100,000	70,000	87,000	77,000	60,000 to 100,000	70,000	77,000	50,000	50,000
Ultimate Stress on Mild Steel, Lb. per Sq. In.			47,300								

TABLE II—AMERICAN TESTS

Ratio of Length to Diameter of Spiral	6	8	9.25	9.25	10	10.3	12	12.9	14
Diameter of Column (Inches)	12	12	6.5	6.5	12	7	12	6.5	14
Number of Columns Tested	2	4	6	2	6	4	2	4	2
l/r of Metal Core	50	66	60	48	83	144	99	84	116
Per Cent of Metal Core	11	11	38	59	11	8	11	38	11
Ultimate Stress on Cast Iron, Lb. per Sq. In.	51,400	42,700	38,500	44,900	41,600	53,500	42,400	33,300	36,500
Ultimate Stress on Mild Steel, Lb. per Sq. In.			36,800			56,300		34,400	

26.5, and the percentage of metal was only 9. Very interesting is the comparison between Columns II and IIIm, which were only 59 in. high. Column II had four structural steel angles and column IIIm four cast iron angles of the same size. Their l/r was only 17.5 and the cores ought to have been stressed nearly to their ultimate strength. Indeed it was found that the structural steel angles contributed 47,300 lb. per sq. in., and the cast iron angles 100,000 lb. per sq. in. to the ultimate load of the composite columns.

In order to verify Dr. Emperger's tests under American conditions the writer had 18 composite columns made, 12 in. in dia. and from 6 ft. to 14 ft. long, with 0.6 per cent each of spiral and vertical reinforcement and with a center cast iron core of pipe 6 in. in dia. and ¾ in. thick. These columns were tested by the U. S. Bureau of Standards in 1916 and a full description of the tests may be found in the 1917 Proceedings of the American Concrete Institute.[2] A summary is shown in Table 2. The stresses on the metal core were found considerably smaller than Dr. Emperger's tests and were ascribed to the poor quality of the hollow cast iron cores which showed great unevenness in thickness.

In 1918 the writer had a new set of tests made of composite columns at the Armour Institute of Technology. These columns consisted of: two standard hooped concrete columns 7 in. in dia. and 6 ft. long with 1 per cent each of spiral and vertical reinforcement; two columns, as above, with a solid shaft of cast iron 2 in. in dia. in the center; two columns as above with a solid shaft of mild steel 2 in. in dia. in the center; two cast iron bars 2 in. in dia. and 6 ft. long; and two mild steel bars 2 in. in dia. and 6 ft. long.

The hooped concrete columns failed at 5010 lb. per sq. in., the cast iron bars tested alone failed at 18,775 lb. per sq. in., the steel bars at 20,375 lb. per sq. in., while the metal cores in the composite columns contributed 53,500 and 56,300 lb. per sq. in., showing a very large stiffening effect, which must be ascribed to the comparatively small percentage of metal reinforcing.

[2] A. C. I. *Proceedings*, Vol. 13, p. 22.

TABLE III—TESTS ON EMPERGER COLUMNS —1926—MADE BY THE AUSTRIAN CONCRETE COMMITTEE

	Mark	Length In.	Dia. of Spiral In.	Ratio Length to Dia. l/d	Area of Core Sq. In.	Vertical Reinforcement 4 Bars .63 In. Ø Area Sq. In.	Vertical Reinforcement 4 Bars .63 In. Ø Per Cent	Area of Structural Steel Sq. In.	Area of Structural Steel Per Cent	Area of Cast Iron Sq. In.	Area of Cast Iron Per Cent	Spiral Reinforcement Size of Wire In.	Spiral Reinforcement Pitch In.	Spiral Reinforcement Per Cent	Mix Cement to Sand Plus Stone	Age. Days	Cylinder Strength Lb. Per Sq. In.	Ultimate Load on Column Total Lb.	Ultimate Load on Column Lb. Per Sq. In.	Load Considered Carried by Hooped Concrete and Vertical Steel Bars Values of Lb. Per Sq. In. Derived From	Load Considered Carried by Hooped Concrete and Vertical Steel Bars Total Lb.	The Remainder of the Load Carried by Structural Steel or Cast Iron Total Lb.	The Remainder of the Load Carried by Structural Steel or Cast Iron Lb. Per Sq. In.
Hooped Concrete Columns with 4 Vertical Bars and Cast Iron Cores	IIIa	59.5	12	4.96	113.5	None	None	9.93	8.75	.197	1/2	2.24	.57	1:6 1/2	24	971,000	8,550	2,300	I_a	238,000	733,000	73,900
	b	59	"	4.92	"	1.245	1.10	"	"	"	"	2.085	.60	"	"	1,032,000	9,100	2,640	"	274,000	758,000	76,600
	c	59	"	5.07	110.0	"	1.13	"	"	"	"	1.97	1.00	"	"	1,138,000	10,000	3,174	"	329,000	808,000	80,200
	d	59	11.63	5.07	106.0	"	1.18	"	"	"	"	2.085	.60	"	14	1,010,000	8,900	1,600	I_a	166,000	844,000	85,000
	e	59.5	12	4.96	113.5	"	1.10	"	"	"	"	2.05	.61	"	24	1,085,000	9,560	2,650	I_c	274,000	811,000	81,700
	f	59	"	4.92	"	"	1.10	9.37	8.25	.275	1/2	1.181	1.65	1:8	69	816,000	7,190	3,575	I_c & d	318,000	827,000	83,300
	g	40.5	11.24	8.06	99	None	None	10.25	10.35	.197	"	2.01	.67	1:4 1/2	"	1,067,000	10,770	3,000	"	266,000	801,000	78,200
	i	"	11.03	8.20	95.5	"	"	4.78	5.00	"	"	2.32	.59	1:6 1/2	24	640,000	6,700	2,220	I_c	201,000	439,000	91,800
	k	"	"	"	"	1.245	1.305	"	"	"	"	2.20	.62	"	"	707,000	7,400	2,564	"	233,000	474,000	99,100
	m	59	11.63	5.07	106.0	"	1.18	9.93	9.37	.275	1/2	1.97	1.02	"	14	1,213,000	11,450	2,200	I_a	211,000	993,000	100,000
	p	40.5	12	7.54	113.5	"	1.10	"	8.75	"	"	1.18	1.65	1:8	69	1,242,000	10,950	3,575	I_f	370,000	872,000	87,900
	q	"	"	"	"	"	"	"	"	"	"	1.26	1.54	1:4 1/2	57	1,544,000	13,600	5,000	I_c	518,000	1,026,000	103,500
	r	"	11.85	7.64	110.0	"	1.13	"	10.3	"	"	2.24	.88	1:6 1/2	24	944,000	8,580	3,015	I_c	302,000	642,000	64,700
	s	"	11.24	8.06	99	"	1.25	"	10.0	197	1/2	2.05	.66	"	21	960,000	9,700	2,700	"	241,000	719,000	72,500
	t	"	12	7.54	113.5	"	1.10	"	8.75	"	"	1.70	.76	1:4 1/2	57	1,169,000	10,300	3,500	I_b	570,000	599,000	60,400
Standard Hooped Concrete Columns with 4 Vertical Steel Bars	II	59	12	4.92	113.5	"	1.10	9.37	8.25	.275	1/2	1.181	1.65	1:8	69	816,000	7,190	3,575	I_f	373,000	443,000	47,300

4 1/8" x 3.14 in. x .39 in. tied 10 in. c. c

To enable the engineers of the new building Code Committee of the City of Chicago to make a comprehensive ruling on composite columns, and especially to find the carrying capacity of metal cores having a large section in comparison with the hooped concrete area, the writer had a new set of columns made on July 13, 1930. These were tested by Prof. W. J. Putnam in the Laboratory of the University of Illinois, in the presence of members of the staff of the University and the Chicago committee. Fourteen columns were tested, having an outside diameter of spiral of 6½ in., the spiral being made of No. 12 high carbon wire at ½ in. pitch. The vertical reinforcement consisted of four ½-in. high carbon bars. The spiral amounted to 1.075 per cent and the vertical bars to 2.35 per cent of the core area. In addition, six columns had 4-in. cast iron cores, representing 38 per cent of the spiral area; two columns had 5-in. cast iron cores, representing 59 per cent of the spiral area; and four columns had 4-in. mild steel cores, representing 38 per cent of the spiral area.

The columns were 5 ft. and 7 ft. long, and two of the cast iron cores were spliced in the center of the height by means of thin wrought iron pipes 5 in. in dia. and 10 in. long. The concreting of the columns was done by foundry workers and the workmanship was not better than that found on the average job. The metal cores were squared at the ends, but were not placed quite square or quite plumb in the forms and it was the intention to have the column ends squared after the concrete was 2 weeks old. This was found impracticable with the tools at hand and the columns were tested with ends in which the concrete was not always even with the metal cores. The thickness of the concrete between metal core and the spiral was only ¾ in. to 1¼ in., which made tamping difficult and voids in the concrete could not be altogether eliminated. Therefore in estimating the load considered to be carried by the metal core, the load carried by the hooped concrete was taken as 5000 instead of the 5750 lb. per sq. in., which was the average strength of the two standard hooped concrete columns tested.

The tests of the spliced cores gave higher results than those of unspliced cores, hence we can say that splices as used do not diminish the strength of composite columns. The same result

was found with spliced cores in some of Emperger's tests and in those of the U. S. Bureau of Standards. None of the cast iron cores failed by breaking. The columns failed by deflection and at the ultimate load some spiral wires were broken. Again the ultimate stresses on the cast iron cores were lower than in the European tests and it is now clear that the hard grade of cast iron used there is more suitable than the soft machinery cast iron obtainable here. The results of the tests are given in Table 4 and are summarized in Table 2. All the tests mentioned show a considerable range of results and it is very difficult to formulate a rule which fits all conditions. It can be positively stated, however, that Dr. Emperger's claim of the strength of the composite columns being greater than the sum of its components has been verified by every test. It can also be affirmed that for columns of $l/d < 6$ and percentages of metal cores < 12 per cent, the stresses on cast iron cores are always greater than 50,000 lb. per sq. in. on common grade of cast iron, and greater than 40,000 lb. per sq. in. on structural steel. For columns having an $l/d = 8$ to 12 and metal cores less than 12 per cent, the ultimate stresses on cast iron equals 40,000 to 53,000 lb. on common grade, and 50,000 to 100,000 lb. per sq. in. on hard grade material. For columns having an $l/d = 14$, the ultimate stresses on the cast iron cores were found to be 36,600 lb. per sq. in. The tests further clearly show that the l/r of the metal core has a large but not preponderant influence on the result. Where l/r was smaller than 30 we always find stresses near the cubical strength of the material, and for l/r smaller than 50 the ultimate stresses on the cores are still very high. On the other hand for percentages of the metal core smaller than 9 (not very important in composite columns) the concrete exerted such a large stiffening effect, especially where l/d was smaller than 9, that the stresses in the metal cores were still found very high for very large values of l/r. The stiffening effect of the concrete is still clearly seen in composite columns with 38 per cent of metal core. While Tetmayer's formula would lead us to expect an ultimate strength of the cast iron of 35,200 lb. per sq. in. for a l/r of 60, the tests showed 38,500 lbs., and for an l/r of 48 we obtained 44,900 lb. per sq. in. against 44,200 according to Tetmayer.

The 4-in. mild steel cores showed stresses of 36,800 and 34,400 lb. per sq. in. as compared with 34,500 and 32,000 lb. per sq. in. by the formula. In nearly all cases common grade of cast iron proved more efficient than structural steel. Where stresses in the cores were less than 50,000 lb. per sq. in., the first cracks always occurred at 70 to 80 per cent of the ultimate loads. At the U. S. Bureau of Standards laboratory, elaborate extensometer tests were made and it was found that the vertical steel bars were stressed to the elastic limit long before the ultimate load was reached, and that the unit deformation of steel and concrete at ¼ load was 0.00035.

The present rulings on composite columns were not based on the tests above mentioned. The Chicago ruling under which quite a number of large buildings have been constructed with composite columns allow the following design stresses:

Stress on hooped concrete of 1:1:2 mix with 1 per cent of spiral ..... 800 lb. per sq. in.
Stress on vertical bars ..... 14,000 lb. per sq. in.
Stress on cast iron cores ..... 10,000 lb. per sq. in.

Detroit Ruling:

Stress on hooped concrete, 1:1:2 mix, with 1 per cent each of spiral and vertical reinforcing ..... 1,120 lb. per sq. in.
Stress on cast iron cores ..... 11,200 lb. per sq. in.

Joint Committee:

Stress on 1:1:2 concrete with one per cent spiral ..... 1,250 lb. per sq. in.
Stress on vertical bars ..... 7,500 lb. per sq. in.
Stress on steel cores not to exceed ..... 15,000 lb. per sq. in.
Stress on cast iron cores not to exceed ..... 9,000 lb. per sq. in.

The cast iron cores are limited to 12 per cent.

From the tests made at the Armour Institute and at the University of Illinois it is clear that the Joint Committee ruling favors structural steel and underestimates the value of cast iron, and on the other hand allows too low stresses on the vertical bars. The limitation of the cast iron cores to 12 per cent is wrong. The writer would propose stresses on the concrete as per ruling of the Joint Committee, stresses of 10,000 lb. per sq. in. on vertical bars, stresses on the metal cores, depending on the ratio of length to diameter as follows:

For l/d —	6	8	10	12	14	16
Permissible unit stress on metal core	13,000	12,000	11,000	10,000	9,000	8,000

TABLE IV

TESTS OF COMPOSITE COLUMNS MADE AT THE UNIVERSITY OF ILLINOIS, JULY 23, 1930, IN BEHALF OF L. J. MENSCH

Number of Column	Fig.	Length of Column, Ft.	Ratio of Length to Diameter	Size of Solid Round Metal Core In.		Area of Spiraled Core, Sq. In.	Area of Metal Core, Sq. In. Percentage	Actual Concrete Area, Sq. In.	Load Considered Carried By		Load Carried by Concrete and Four ½-in. Ø, 1000 Lb.	Actual Test Load in 1000 Lb.		Load Carried by Metal Core		Average Load Carried per Sq. In. of Spiraled Core-Area, Lb.	Load at First Crack, 1000 Lb.
				Cast Iron	Mild Steel				Concrete at 5000 Lb. per Sq. In., 1000 Lb.	Four ½-in. Ø at 4000 Lb. per Sq. In., 1000 Lb.		P	Average	Total 1000 Lb. Average	Lb. per Sq. In.		
1	2	5	9.25			33.2	No	M	etal	Co	re	255				7,680	
2		7	12.9									193				5,810	
3	3	5	9.25	4		33.2	12.6	19.8	94	32	12.6	606	611	485	38,500	18,400	485
4							38					616					470
5	3	7	12.9	4		33.2	12.6	19.8	94	32	12.6	525	545	419	33,300	16,400	490
6							38					565					
7	3	5	9.25	4		33.2	12.6	19.8	94	32	12.6	646	636	510	40,500	19,150	
8							38 Spl	iced	Cen	ter		626					
9	4	5	9.25	5		33.2	19.6	12.8	64	32	96	980	988	892	44,900	29,800	
10							59					997					740
11	3	5	9.25		4	33.2	12.6	19.8	94	32	12.6	591	589	463	36,800	17,750	550
12							38					588					
13	3	7	12.9		4	33.2	12.6	19.8	94	32	12.6	557	559	433	34,400	16,850	530
14							38					561					525

All spirals consisted of No. 12 with ½ in. pitch. Area of spiral core, 33.2. Equiv. area of spiral, 1.075 per cent of spiral core. Area of four ½-in. Ø = .784 sq. in. = 2.35 per cent of spiral core.

Concrete mixture 1:1:2. Columns were 40 days old when tested.

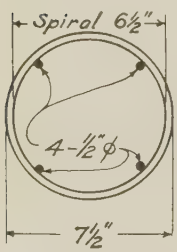


FIG. 2

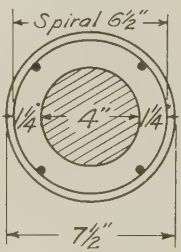


FIG. 3

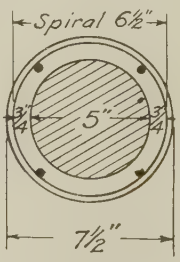


FIG. 4

which would result in factors of safety of 3.5 to 5 depending on the percentage of metal reinforcement used and on the l/r of the core. Composite columns have shown great uniformity of results where duplicate tests have been made, and they are just as reliable as steel columns. Hence the proposed factor of safety is very conservative in comparison with that of steel columns which is generally only 2½ and sometimes less.

As a result of these tests there are these observations:

High carbon wire for spirals was found much superior to ordinary steel wire, and the increase in strength of the hooped concrete was found proportional to the elastic limit of the hooping wire, or the ultimate strength in case of hard wire. A close pitch of the spiral wire produced more uniform results than a large spacing, and is very important in composite columns because the thickness of the concrete between core and spiral often is only a few inches. The practice of using a great number of vertical bars, 8 or more, to prevent a flow of the highly stressed hooped concrete is not warranted by these tests, and the flow is more positively prevented by a small pitch.

Dr. Emperger has used composite concrete sections of rectangular shape for ribs in a number of important arch bridges, and had tests made on rectangular columns with rectangular spirals of close pitch, containing about 10 per cent of cast iron core. He tested a number of sections of 9-in. x 19-in., having a kind of I-section of cast iron with large openings in the web, the cast iron representing 11 per cent of the section. The l/d was 14.4, the l/r was 84 and the ultimate load per sq. in. which the cast iron core supported was figured at 42,000 lb. The same sections loaded with an eccentricity of ⅓ the depth supported a load of ⅔ the concentrical load. Such composite compression members are of great advantage in arched bridges of great spans. Instead of plain reinforced concrete ribs of an ultimate strength of about 3000 lb. per sq. in., composite ribs can be designed for 18,000 lb. per sq. in. and more. The lighter section of the ribs not only reduces the cost of the abutments and that of the form work, but very materially decreases the stresses due to shrinkage or a drop in temperature. Deflections and settlements of the abutments cause very much smaller stresses in small crown sec-

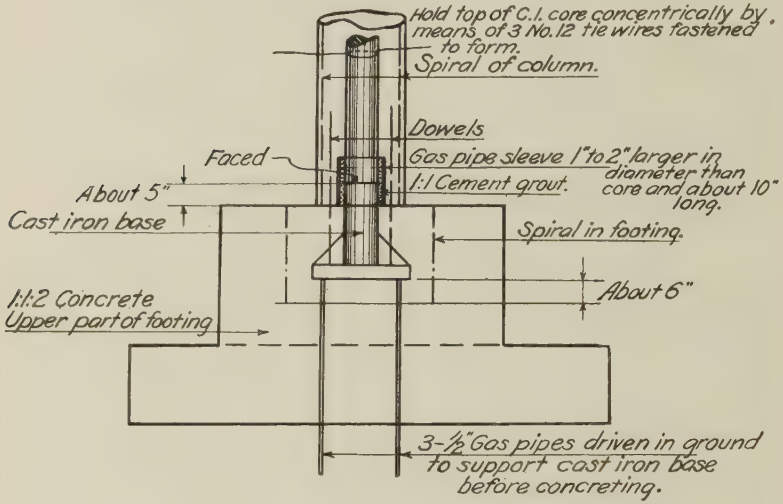


FIG. 5—TYPICAL FOOTING FOR COMPOSITE COLUMN

tions than in large ribs and make the introduction of hinges almost superfluous. Engineers are enabled to use arch bridges in localities where rock foundations hitherto considered necessary for every arch bridge, are not available for the abutments. It is also an easy matter to design the metal core for the support of the suspended formwork, thereby reducing the cost of this most important item to perhaps one third. Dr. Emperger has built a number of bridges from 200 to 300 ft. span in competition with wooden bridges and nearly all his bridges had abutments supported by piles. He obtained patents for his system of composite sections in nearly all civilized countries. The author has made designs for heavy bridges up to 1000 ft. span and found their cost considerably lower than steel or common concrete bridges of the same total length and divided into two or three spans.

In the practical design of composite columns the load from the metal core is transmitted to the footing by means of a cast iron base, as shown in Fig. 5. The upper part of the footing is made of 1:1:2 concrete and is strengthened by a spiral of about 4½ times larger than the core. The load from the core is transmitted

TABLE V

COMPARATIVE DESIGN OF EMPERGER COLUMNS AND STANDARD HOOPED CONCRETE COLUMNS FOR A TALL BUILDING ACCORDING TO THE CHICAGO CODE.

Emperger Columns					Standard Hooped Concrete Column		
Load in 1000 lbs.	Side of Sq. Col.	Per cent of Spiral	Verticals	Dia. of Solid Cast Iron Core	Side of Sq. C.	Per cent of Spirals	Verticals
22nd Story 100	25″	ties	4— ⅞″ rd.		18″		4— ¾″ rd.
160	25″	ties	4— ⅞″ rd.		18″	½%	8— ⅞″ rd.
220	25″	ties	4— ⅞″ rd.		18″	1½%	8—1″ sq.
280	25″	½%	8— ¾″ rd.		22″	1½%	8—1″ rd.
340	25″	1 %	8— ¾″ rd.		22″	1½%	8—1¼″ rd.
400	25″	1½%	8—1 ″ rd.		22″	1½%	11—1¼″ sq.
460	25″	1½%	8—1¼″ sq.		26″	1½%	8—1¼″ sq.
15th Story 520	25″	1½%	10—1¼″ sq.	2½″	26″	1½%	13—1¼″ sq.
570	25″	1½%	10—1¼″ sq.	3½″	26″	1½%	16—1¼″ sq.
630	25″	1½%	10—1¼″ sq.	4½″	30″	1½%	10—1¼″ sq.
690	25″	1½%	10—1¼″ sq.	5½″	30″	1½%	12—1¼″ sq.
745	25″	1½%	10—1¼″ sq.	6″	30″	1½%	16—1¼″ sq.
802	25″	1½%	10—1¼″ sq.	6⅝″	30″	1½%	20—1¼″ sq.
870	25″	1½%	10—1¼″ sq.	7⅝″	30″	1½%	24—1¼″ sq.
930	25″	1½%	10—1¼″ sq.	8″	34″	1½%	16—1¼″ sq.
985	25″	1½%	10—1¼″ sq.	8⅝″	34″	1½%	20—1¼″ sq.
1020	25″	1½%	10—1¼″ sq.	8⅝″	34″	1½%	23—1¼″ sq.
1070	25″	1½%	10—1¼″ sq.	9″	34″	1½%	26—1¼″ sq.
1152	25″	1½%	10—1¼″ sq.	9⅝″	38″	1½%	20—1¼″ sq.
1215	25″	1½%	10—1¼″ sq.	10″	38″	1½%	24—1¼″ sq.
1280	25″	1½%	10—1¼″ sq.	10¾″	38″	1½%	29—1¼″ sq.
1st Story 1350	25″	1½%	10—1¼″ sq.	11″	38″	1½%	34—1¼″ sq.
Basement 1420	25″	1½%	10—1¼″ sq.	11⅜″	38″	1½%	43—1¼″ sq.

From above data it is found that, assuming all columns to be 11 ft. high in each story,

Emperger columns contain:
1,100 cu. ft. of 1:1:2 concrete,
3,114 lb. of spirals
13,750 lb. of vertical steel
2,112 sq. ft. of forms, and
29,293 lb. of cast iron

Standard columns contain:
1,574 cu. ft. concrete
7,255 lb. of spirals
29,535 lb. of vertical steel
2,460 sq. ft. of variable forms.

by bond on the shaft of the base and by direct compression of about 1000 lb. per sq. in. on the plate. The direct stresses from the hooped concrete of the basement column can be assumed to spread out in the shape of a cone between the base plate and the footing spiral. Before the cast iron base is set, three ½-in. gas pipes are driven into the ground to the correct elevation of

the under side of the plate and the base is lowered to the proper level. The shaft of the base is made long enough to project 5 in. above the finished footing, and after the footing is concreted a gas pipe sleeve, one or two inches larger in diameter than the core, is placed concentrically over the protruding shaft and filled with cement grout of 1:1 mix to about ¼ in. below the top of the shaft. After the basement column form and the spiral and vertical rods are in place, the gas pipe sleeve is partly filled with grout by pouring from the top of the form and the metal core is lowered into the sleeve by means of a gin pole, the grout acting like a mirror, thus enabling the setter to center the core. Sometimes ¼ in. rods are placed in the space between the core and the sleeve to facilitate centering. The grout fills the space between sleeve and core, and to assure that no gravel has lodged between the faces of the cores, the upper core is turned a few times to the right and left to crush it. The top of this core projects 5 in. above the first floor and the connections on each floor are made with sleeves as described for the basement story. The top of the core is held in place by 3 wires connected either to the columns or floor forms.

It is important for engineers to know what size column to adopt to obtain an economical and practical design. They have to be guided by the fact that a great saving in the cost of the entire skeleton can be effected by making all columns of each column stack of the same size, and if possible use only one or at the utmost two or three column sizes in the entire structure. It is very much easier for the contractor to build all his column forms on only one or two benches without change of templates; he will not need special foremen to superintend the rebuilding of forms story by story. Neither will he have to rebuild the girder and lintel forms. He will need only two sets of column forms and not more than two sets of lintel forms even if his progress is at the rate of three stories per week, because he does not lose any time for rebuilding. Hence less talent has to be spent on the form work and more time is left for the leaders to see that good work is done on the concreting, and competent contractors will soon learn that they can do the form work for columns, girders and lintels for at least 5¢ per square foot less than in the old system with its repeated changes of form.

Experience has shown that a composite column stack with the same size of columns from top to bottom, costs less than a column stack with standard hooped concrete columns of varying sizes, when the basement columns are designed for about 4000 to 5000 lb. per sq. in. on the area within the spiral hooping. Table 5 shows such a comparative design for a 22-story building which has been constructed in the City of Chicago.

When the composite columns are made larger the cost is less, and when they are made smaller than shown in Table 5 their cost may become slightly higher than that of standard hooped concrete columns.

In order to solve the question of bond of the cast iron core with the hooped concrete, the writer had tests made by Prof. D. A. Abrams on smooth 2 in. bars 12 and 24 in. long, embedded in a 7-in. diameter hooped concrete block. The results of the tests were not published. The bond stress at failure was found to be from 900 to 1200 lb. per sq. in., and confirmed similar results found by Considere on gas pipes embedded in hooped concrete.[3] The bond stress is so much higher than in plain concrete because the hoopings prevent the concrete from splitting up. This high bonding value enables the metal core to take up in the depth of a girder the heaviest loads any girder or number of girders ordinarily transmit to a column in any one story; hence brackets or rings on the core beneath the girders are unnecessary. Engineers often inquire how the reinforced girders are connected to the columns. There is hardly any difference between this type of column and the ordinary concrete column, especially as the modern tendency is to make girders not much deeper than the joists framing into them. This requires that the girders be made wide, preferably as wide as the columns for economy in form work, which enables most of the straight and bent bars to pass by the cores, as there is nearly always at least 5 in. of concrete outside the cores. In special cases it may be necessary to use two or more extra bars for the negative moments at the columns. In a number of cases steel girders have been connected to brackets on the cores and

[3]Commission du Ciment Armé. Rapport etc. 1907.

in one case trusses, carrying over a million pounds, have been set on top of the cores.

It is the writer's practice to use as longitudinal reinforcement from 2 to 4 per cent of high carbon bars to take care of any bending moment which may be exerted on the columns. Because the loads are very large on composite columns, the eccentrity produced by connecting girders is not as a rule of great amount. It is different, however, with columns forming part of a wind

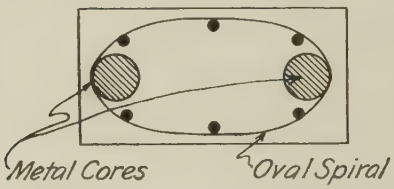


FIG. 6—SECTION OF COMPOSITE COLUMN FORMING PART OF WIND BRACING SYSTEM

bracing system. There the eccentricity of loading may be as large as 6 to 9 in., and such columns act similarly to the ribs in an arch bridge, and a design as shown in Fig. 6 will be more economical in buildings higher than 20 stories. Nearly all columns tested by the writer showed eccentricities of loading which varied from 1/20 to 1/10th of the diameter of the spiral; hence, engineers need not be too anxious about small eccentricities.

A comparison between composite columns with cast iron cores and structural steel cores might be of interest to the profession. Let us assume a column 15 ft. high has to support a load of 2,000,000 lb. Adopting the rule proposed by the writer we shall allow for 1:1:2 concrete with 1 per cent spiral, a stress of 1250 lb. per sq. in., and 10,000 lb. minus 1250 lb. or 8750 lb. per sq. in. on the vertical steel. Adopting a spiral of 22 in. we find the l/d equals about 8, hence the permissible stress on the core is 12,000 lb.—1250 lb. or 10,750 lb. per sq. in. The area of the 22-in. concrete core is 380 sq. in. and the area of eight 1¼-in. square bars, representing about 3 per cent of the area inside the spiral is

12.5 sq. in., hence the load carried by the hooped concrete is 380 x 1250 lb. = 475,000 lb.; by the vertical steel 12.5 x 8750 lb. = 109,000 lb., giving a total of 574,000 lb. Hence the metal core has to carry a load of 2,000,000 lb. — 574,000 lb. = 1,426,000 lb., requiring a sectional area of 1,426,000 divided by 10,750, or

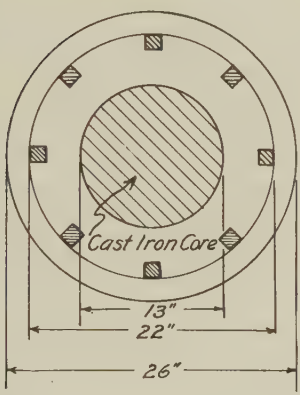


FIG. 7—SECTION OF COMPOSITE COLUMN WITH CAST IRON CORE

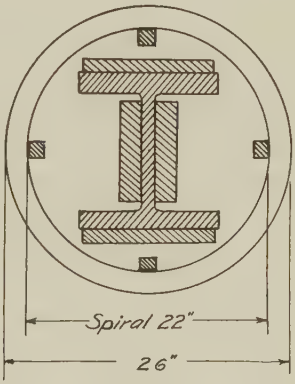


FIG. 8—COMPOSITE COLUMN WITH STEEL CORE OF BETHLEHEM H-COLUMN WITH COVER AND WEB PLATES

132.5 sq. in., equivalent to a cast iron core exactly 13 in. in diameter. The l/r will be about 55, and we may expect this core to fail at a stress of about 50,000 lb. per sq. in., if made of common grade of cast iron. This cast iron core will weigh about 425.0 lb. per lin. ft. A Bethlehem 16-in. H-column section would have about 126 sq. in. but cannot be used in any spiral less than 28 in. in dia., hence we have to use a 12 in. H-section weighing 190 lb. per lin. ft. and reinforce it with two cover plates 12 in. x 1¼ in. and two web plates 9 in. x 2 in., giving a total area of 121 sq. in. and weighing about 435 lb. per lin. ft. This section has about 9 per cent less area than the cast iron core, but is heavier, and costs, of course, a great deal more. Both sections are shown in Figs. 7 and 8. While cast iron cores can be shipped within 3 days, deliveries of structural steel cores would require

a great deal more time. If composite columns come into vogue it will, of course, be possible to get round sections rolled on short order.

An inspection of Fig. 7 will show that composite columns are more nearly fireproof than fireproofed steel columns because there is as a rule at least 5 in. of concrete outside of the metal core, and the core alone can support the load with a good factor of safety, even if the concrete is entirely destroyed. Composite columns have also been used in low buildings where small sizes of columns are of importance, such as garages and certain manufacturing buildings, or where the contractor appreciates the saving of form work in having all columns in the building of the same size. Engineers, also, ought to appreciate the savings in their own time of preparing column schedules. When all columns are the same size, and spirals and vertical steel are alike, the schedule will have to show only the size of the core for each story, and a simple note will take care of the spiral and vertical rods for most of the columns.

Readers are referred to the JOURNAL *for March 1931 for discussion which may develop. Such discussion should reach the Secretary by February 1, 1931.*

Specifications For Ready-Mixed Concrete

Proposed American Concrete Institute Specification No. 504

MILES N. CLAIR,* AUTHOR-CHAIRMAN

Committee 504, Specifications for Ready Mixed Concrete

1. General

(A) These specifications cover the special conditions affecting the use of Ready-Mixed Concrete and supplement the requirements of the General Specifications for Plain and Reinforced Concrete for this Contract.

(B) The term "Ready-Mixed Concrete" is used in these specifications as meaning[1] Central Plant-Mixed concrete and Central Plant-Proportioned Truck-Mixed concrete.

(C) The place where the materials are proportioned is referred to in these specifications as "the plant."

(D) The operation of the plant shall be under the direct supervision of a competent engineer.

2. Materials

(A) All aggregates, cement and water shall be of the quality required by the General Specifications for Plain and Reinforced Concrete. All conveyors, bins and hoppers shall be cleaned of unapproved materials before starting to manufacture concrete for this contract.

(B) No change shall be made in the materials used, during the course of this contract, except with the permission of the Architect or Engineer.[2]

3. Concrete quality and proportioning

(A) The concrete quality and proportions shall be in accordance with the requirements of the General Specifications for

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[1]The Architect or Engineer must indicate which systems are allowed. It is recommended that in general both be included.

[2]In the use of this specification for a particular project one of these must be stricken out throughout the text to avoid conflict.

Plain and Reinforced Concrete. Proportions, however, shall be subject to change under the direction of the Architect or Engineer when necessary to produce proper workability or satisfactory strength.

(B) The measurement of the concrete aggregates and the cement[3] shall be by weight so that the proportions can be readily adjusted to assure a definite volume of concrete of the quality specified. The measurement of the water may be by weight or volume. The volume of concrete produced per batch shall be taken as the sum of the absolute volumes of the ingredients.

(C) A statement of the control quantities[4] used and the time at which the vehicle was loaded shall accompany each load of concrete. The organization supplying the ready-mixed concrete shall have available sufficient test data, certified by a competent concrete testing engineer, to enable the Architect or Engineer to determine whether the quantities used will give concrete of the desired quality and the stated quantity per batch.

(D) Facilities shall be provided at the plant to enable the Architect or Engineer to check easily at any time the quality of the materials and the accuracy of the measurements.

(E) Tests of the concrete as delivered to the forms shall be made during the progress of the work as required by the General Specifications for Plain and Reinforced Concrete.

4. MIXING AND TRANSPORTING CONCRETE

(A) *General*

(1) Transporting vehicles shall be so constructed as to insure rapid delivery without loss of ingredients.

(2) All concrete shall be mixed in a revolving drum type of mixer, whether stationary or movable. The mixing time or number of revolutions shall be as recommended by the manufacturer; but shall not be less than required by the General Specifications for Plain and Reinforced Concrete. The mixer shall not be loaded beyond the capacity recommended by the manufacturer.

[3]Bag measurement may be used provided the batches do not involve fractions of bags.
[4]"Control quantities" refers to the requirements of the specifications such as water-cement ratio, cement content, or proportions.

(3) A maximum period of 00 hours[5] may elapse between the addition of the cement to the aggregate and the placement of the concrete in the forms; but the concrete as placed shall be of the consistency and workability required by the General Specifications for Plain and Reinforced Concrete.

(4) Under no circumstances shall partially hardened or retempered concrete be placed in the structure.

(5) Special precautions shall be taken to protect the materials from damage during transportation under extreme weather conditions.

(B) *Central Plant-Mixed Concrete*

(1) The concrete shall be properly mixed in accordance with the requirements of the General Specifications for Plain and Reinforced concrete before being loaded into the transporting vehicle.

(2) Non-agitating type transporting vehicles shall not be used to transport Central Plant-Mixed concrete when the consistency of the concrete is wetter than a —-inch slump.[6]

(3) Remixing type transporting vehicles[7] shall not be used to transport Central Plant-Mixed concrete when the consistency of the concrete is wetter than a —-inch slump.[6]

(4) Agitating type transporting vehicles may be used without restriction as to consistency.

(C) *Central Plant-Proportioned Truck-Mixed Concrete*

(1) The procedure in loading the truck mixer[8] shall be such that the cement and aggregates are uniformly distributed throughout the batch when the loading is completed.

(2) Where the concrete is mixed during transportation of the batch to the job, means shall be provided so that the mixing time

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[5]The Architect or Engineer must indicate the limiting period. A maximum period of one hour is recommended. (The committee is not unanimous. One group approves of the limit of one hour, as given. The remainder of the committee approves a statement in the footnote referring to this item as follows: "Present knowledge of the art indicates that the time may be as long as three hours under favorable conditions. It should be limited by the engineer or architect after a study of job conditions, kind of aggregates, and the prevailing temperature and weather conditions at the time the structure is being erected.")

[6]The Architect or Engineer must indicate the limiting slump. A maximum allowable slump of 2½ ins. for non-agitating type trucks and 6 in. for remixing type trucks is recommended.

[7]Re-mixing vehicles are those provided with special means of remixing or agitating the concrete to some degree during discharge but not during transportation.

[8]It is suggested that the cement and aggregates be fed simultaneously into the end loading type of truck mixer. For the top loading type of mixer this method of charging is not so essential; but is also recommended.

and the quantity of water added can be readily verified by the Architect or Engineer.

5. DELIVERY

(A) The organization supplying concrete shall have sufficient plant capacity and transporting apparatus to insure continuous delivery at the rate required.

(B) The interval between loads shall not exceed — minutes,[9] or in any case be so great as to allow the concrete in place to become partially hardened.

(C) The methods of delivering the concrete shall be such as will facilitate placing with the minimum of re-handling and without damage to the structure or concrete.

DISCUSSION

BY THE AUTHOR-CHAIRMAN

The specifications for Ready-Mixed concrete presented herewith to the Institute for tentative adoption have been developed after considerable discussion, among the members of the committee and others of the proposed specification published in the JOURNAL for February, 1930.* The questions about which the discussion centered are:

(1) Should the specification be considered as a complete specification in itself or as a supplement to a general specification for plain and reinforced concrete?

(2) Should separate specifications be developed for Central Plant-Mixed Concrete and for Central Plant-Proportioned Truck-Mixed Concrete?

(3) Should special limitations be placed on the methods of measuring materials?

(4) What limitations should be set up for the use of the various types of transporting vehicles and mixers?

The decision of the majority of the committee is apparent from the specifications here proposed; but some discussion and explanation may be of value in connection with its proper use.

*A. C. I. *Proceedings*, Vol. 26, p. 467.

[9]The Architect or Engineer must indicate the interval allowed. A maximum of thirty minutes is recommended.
(The committee is not agreed as to the consistency or slump limitation. The group in opposition to the form proposed desires something as follows in the footnote. "The Architect or Engineer must indicate the limiting slump. Practice indicates that while there are definite limitations to this type of transportation, the limiting slump will be determined by the particular type of equipment used, the nature of the mix, the length of haul, and the character of the road over which it is to be hauled."

Most Architects or Engineers use a standard general specification for plain and reinforced concrete based on the Joint Committee Report. That specification is relatively complete in regard to requirements for materials, proportions, mixing, quality of concrete, placing of concrete, and other items. The requirements necessary under most of these items for Ready-Mixed concrete are no different to secure good concrete than for job-mixed concrete. The Architect or Engineer is still conservatively inclined to use job-mixed concrete rather than the recently developed ready-mixed concrete; and, therefore, a specification that supplements his present general specification with additional safeguards has special appeal. The additional complication of separate specifications for central-plant-mixed concrete and central-plant-proportioned-truck-mixed concrete seems at this time unnecessary. Possibly abuses or future developments may later warrant such a separation.

The specification proposed is, as stated in its first paragraph, intended for use as a supplement to the General Specifications for Plain and Reinforced Concrete for a particular contract; and, therefore, only such special requirements are given as appear necessary to insure concrete of the quality desired. The conception of a specification as a statement of the results desired rather than of the methods or equipment to be used is adhered to as far as possible. It is considered the function of another department of the Institute to set up requirements for equipment and procedure to meet these specifications. The specifying of the supervision of the plant operation by a competent concrete engineer is considered necessary because the plant must be ready to produce on almost instant demand concrete to meet any one of many specifications. A technical education combined with considerable field experience with concrete construction and the control of concrete quality are essential requirements for such a supervising engineer.

Two special requirements are set up in regard to materials. The ready-mixed concrete plant in general has a tie-up with definite material sources which will satisfy the usual specifications. Special materials are required for some work, however, and contamination by usual materials must be prevented. Like-

wise, change of source of materials during the course of the work must be subject to approval by the Architect or Engineer so that not only the quality but also the appearance will not vary.

Although most general specifications for plain and reinforced concrete at present set up requirements in terms of strength, cement content, water-cement ratio, and workability, and do not closely limit proportions, the concrete as produced by the ready-mixed concrete operation may call for changes which the Architect or Engineer should be free to make. The requests for such changes will, however, most frequently come from the plant operator who knows from experience the best combinations, for his equipment and conditions, to meet certain requirements. Changes as requested by the concrete foreman on the job should never be made without the approval of the Architect or Engineer.

A ready-mixed concrete plant cannot accurately meet the demands of numerous specifications and pre-determined quantities unless the aggregates are weighed. The entire operation becomes simplified if all the materials including cement and water are weighed. One of the principal causes of controversy in regard to ready-mixed concrete is the basis on which the volume of concrete delivered shall be measured. Direct measurement in most cases is not possible; but if weight proportioning is used, the yield as computed from the absolute volumes of all the materials is sufficiently accurate and will closely agree with direct measurements.

The Architect or Engineer on the usual construction project can at any time readily go to the mixer and check up on the mix. He also generally has some check on the amounts of materials used, and so will know whether the concrete is nearly of the mix required. The ready-mixed concrete plant is almost always at an appreciable distance from the job; and, therefore, a statement of the basic control quantities used for each load is desirable. These quantities can be used not only as a check on proper quality and quantity, but also to enable the Architect or Engineer to estimate, from the test data and records of batch quantities available at the plant, the probable strength at any given period.

The ready-mixed concrete operation differs from the usual job concrete procedure mainly in the mixing and transporting of the

concrete. The central-plant-mixed concrete system mixes in a regular mixer of a large size, and transports the concrete to the job in ordinary trucks or special trucks. No comments are necessary in regard to the mixers except that they are usually of larger sizes than is common for job set-ups; and, therefore, the mixing time required is generally greater. Minimum mixing times of one minute for a one-yard mixer, 1½ minutes for a two-yard mixer, and two minutes for a three-yard mixer, are recommended. The conveying vehicles should be water-tight so that no ingredients of the mix are lost. Agitator trucks should be designed so that they really do keep the concrete mixed throughout the truck. Overloading of agitator trucks results in decided decrease of efficiency of the mixing action.

Central plant-proportioned-truck-mixed concrete requires special mixing and conveying equipment. Some of the truck mixers are essentially the same as the standard mixers of low length-diameter ratio; but the more common type is of high length-diameter ratio. These truck mixers also range to sizes as large as five cubic yards. Experience has indicated that a mixing time of five minutes, or better 40 revolutions, is necessary for such mixers. There is plenty of time for such mixing during the trip to the job. Some end-loading types of mixer trucks also require special care in loading to assure uniform distribution of the materials, unless excessively long mixing time is used. The truck mixer type of operation puts the mixing and water measuring operations in the hands of the truck driver, if done on the way to the job; and, therefore, some means of readily checking up on these operations should be provided.

The Architect or Engineer is most concerned when using ready-mixed concrete that the concrete comes to him in a homogeneous mix and without damage due to the time elapsing in transportation. Transportation of the concrete in an ordinary truck that may be used at other times for sand and gravel appeals to the ready-mixed concrete plant operator because of its low cost. Experience, however, has shown that concrete wetter than a 2½-in. slump cannot be transported in such trucks without segregation and consequent difficulty in unloading and detriment to the quality of the concrete. Under special conditions of mix, ma-

terials, or very short hauls, wetter concrete has been successfully handled; but such cases are not usual.

Agitator type trucks are satisfactory for concrete of any slump likely to be used. Most agitators keep the concrete well-mixed despite the surprisingly small amount of movement given the mass. Improvement in the design of some agitator trucks is desirable, however, to insure a more uniform mixing action, particularly where several batches are used to fill one truck.

Most architects and engineers agree that concrete that has partly hardened should not be used. What is meant by "partly hardened" is difficult to define; but it is more readily determined in practice, as such concrete has a peculiar crumbly appearance when mixed which is different from concrete that is just dry. The limit to be set on the time elapsing between the addition of the cement to the batch and the placing of the concrete in the work will depend on many factors; and cases have been cited to favor a shorter or longer period. Without tests for a particular set of conditions, it is recommended that a maximum elapsed period of one hour be allowed. This is considered as properly applying to truck mixing operations as well as to central plant mixing, because the water in the aggregates in the former operation is available for action with the cement.

The section of the specification on Delivery is intended to safeguard the work from delay due to failure of the ready-mixed concrete supply. It might in many cases well include a requirement of a bond to insure proper service and to cover damages.

The writer is considerably indebted to the critic members of this committee for their suggestions, criticisms and interest. Unanimous agreement in regard to all items was not possible or expected as the use of Ready-Mixed Concrete is comparatively new. Changes in any specification will be made as experience in the field concerned gives more facts on which to base conclusions. It is hoped that this specification will be considered of sufficient merit to be given a working trial as a Tentative Standard Specification.

Readers are referred to the JOURNAL *for March, 1931, for discussion which may develop. Such discussion should reach the Secretary by February 1, 1931.*

Discussion of Report of Committee 604

"Winter Concreting Methods"*

BY I. E. BURKS AND G. D. DURHAM†

It was not so very many years ago that constructors faced with the necessity of placing concrete in freezing weather, attacked the problem with apprehension and doubt. There seemed to be a feeling that a structure erected under such conditions *might* be satisfactory and a fair chance that it might *not* be. Today, thanks to the information made available through the publications of various technical societies, and through the experiences of individual engineers and constructors, we are able to attack a concrete job at 30 or 40 degrees below zero with a feeling of absolute confidence in the results—*if certain rules of the game are carefully followed.*

It is certainly true that there are a few examples of partial failure, directly attributable to frozen concrete. There are also dozens of noteworthy examples of excellent concrete work done at extremely low temperatures. Committee 604 hopes to be successful in correlating the experience and observation of constructors responsible for the successful operations and also the failures. Mr. Johnson, in his preliminary report, has paved the way for writing recommended practice which may eventually be adopted as standard by the Institute, and which may be followed with confidence and safety by engineers and constructors.

In his report as author-chairman of the committee, Mr. Johnson limits its scope to concrete construction in ordinary buildings —but, he points out that the principles involved are equally appli-

*By Robert C. Johnson, as author-chairman of committee 604, Winter Concreting Methods, published in the Journal, Feb. 1930—A. C. I. *Proceedings*, Vol. 26, p. 397. See also discussion Journal, June 1930—*Proceedings*, Vol. 26, p. 904.

†Mr. Burks is concrete technician for the Aluminum Company of America, member of Committee 604 and wrote "Concreting Methods at the Chute a Caron Dam" (in the Journal for Feb. 1930—A. C. I. *Proceedings*, Vol. 26, p. 315) where concrete was placed at 40° below zero. Mr. Durham is his associate on that work.

cable to other types of construction less difficult to protect. For the purpose of this discussion, perhaps it would be helpful to divide all concrete work into two general classes, viz., Reinforced Frame Sections, and Heavy Mass Work. We agree that the principles of protection are the same for either of the classes named but suggest that the objective may be slightly different. To cure concrete for such a time as simply to protect it from damage by freezing is one objective; to cure it for an additional period in order that it may attain a specified strength at a designated time, is another objective. The first is usually the only requirement in heavy mass work. The second, is generally an important and necessary consideration in reinforced frame construction.

If concrete freezes before it solidifies, the uncombined water forms small ice crystals throughout the mass. If it is thawed and thoroughly cured at temperatures above 50° it will attain a strength equal to from 75 to 95 per cent of its potential strength. It has often been stated that *one* such freezing does little if any permanent damage to the concrete, nevertheless, as pointed out by R. B. Young,[1] it is difficult to conceive that the pores formed by the ice crystals are all entirely closed and the mass consolidated as it would have been had freezing not occurred. From this it is evident that the greatest damage done by freezing occurs after the concrete begins to solidify or harden.

The length of time that concrete should be cured to prevent its being damaged by freezing, depends mainly on four factors: (a) type of cement; (b) richness of mixture; (c) amount of heat supplied by the aggregates and mixing water; (d) temperature of the air within the protective enclosure.

In heavy mass construction, the safe protection rule is to provide means of maintaining the temperature of the green concrete near exposed surfaces, above 50° F. until it has attained sufficient strength to withstand frost action. In general it may be stated that concrete designed to have compressive strength of from 1500 to 2000-lbs. per sq. in. at 28-days should be kept at or above the temperature stated until 40 per cent of the designed strength is developed—2000 to 3000-lb. concrete should have 30 per cent, and 3000 to 4000-lb. concrete should have 20 per cent

[1]Discussion of Report of Committee 604—A. C. I. JOURNAL, June 1930—A. C. I. *Proceedings*, Vol. 26, p. 904.

of its designed strength before the temperature of sections near the surface is allowed to fall below 50° F. In gravity dams, high strength is desirable but it is not essential and curing is largely directed to obtain a surface that will withstand the weathering action of the elements. The surface area is small as compared to the mass and the heat loss by radiation is small —consequently the heat developed by the hardening concrete is retained for a longer period.

In thin reinforced frame work, the concrete may be protected until the danger by freezing is eliminated and still not attain its designed strength at the 28-day period. Consequently, there is a possibility of damage by premature loading unless proper protection is provided to meet the requirements of individual cases.

Small walls and structural members encountered in building construction offer a very definite structural hazard when subjected to freezing. The problem of protection and curing is to enable these members to obtain their designed strength before being loaded. In proportion to their mass the surface is great—hence the heat developed by the chemical reaction of the hydration of the cement plus the heat supplied at the mixer, is readily dissipated by the lower air temperatures. Therefore, in thin reinforced frame construction, the forms must be covered and the enclosure maintained at a temperature sufficiently high and for a period long enough to insure the development of the required strength.

On the work on which the authors are engaged,[2] a series of preliminary tests was made to determine the length of time it is necessary to cure concrete placed at temperatures considerably below the freezing point. The experiments were made by the field laboratory organization in the winter of 1929-30 when it was convenient to take advantage of the low atmospheric temperatures for freezing the specimens. The work had to be temporarily discontinued in the early Spring of 1930 because of temperature conditions, but it is to be resumed in the Winter of 1930-31. The series on which results are available at this time includes only one water-cement ratio—viz., 0.80. The second series is to be made using a water-cement ratio of 1.0.

[2]Chute a Caron Dam, Province of Quebec, Canada.

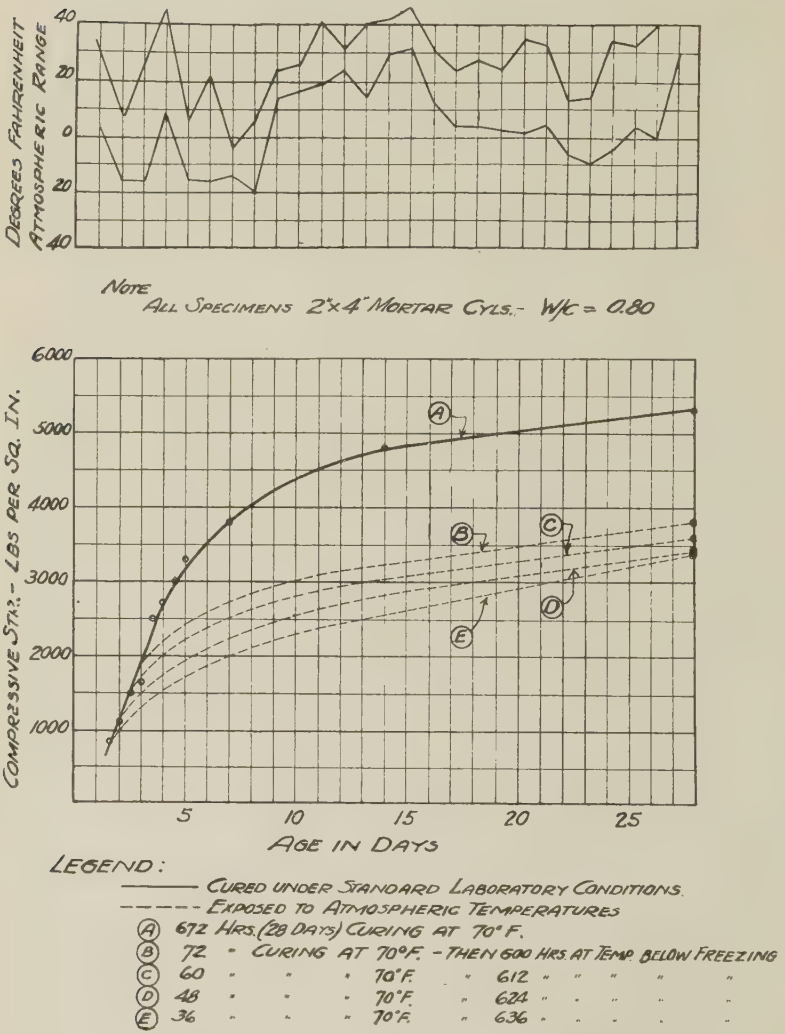


FIG. 1—EFFECT OF EARLY CURING AT LOW TEMPERATURES ON 28-DAY STRENGTH

For convenience, and since comparative results only were required, all specimens made were 2 by 4-in. mortar cylinders, formed in accordance with standard practice. The specimens were cured for varying periods under standard conditions and then exposed to the existing freezing weather. The earliest breaking period was 36-hours. Breaks earlier than this were so low as to be of questionable value and were probably beyond the range of accuracy of the testing machine. All specimens were capped and allowed to remain in laboratory air (70°) one-hour before testing. Figs. 1, 2 and 3 show the results obtained. It should be pointed out that in these experiments the effect of freezing is magnified somewhat because the surface area of the test specimens used is so great in proportion to the volume.

In Fig. 1, Curve A represents the potential strength of a mortar at various ages up to 28-days when cured under standard laboratory conditions. Curves B, C, D and E indicate the strength of the same mortar when cured under standard conditions for 72, 60, 48 and 36 hours and then allowed to freeze for the remainder of the 28-day period. It will be noted that the frozen specimens acquired from 63 to 70 per cent of the potential 28-day strength.

Fig. 2 shows the same general strength relation as Fig. 1, except that all frozen specimens in this group were broken at the age of 7-days. It will also be noted that slightly different temperature conditions prevailed. In this group the specimens were cured at 70° for 36, 48, 60 and 72 hours and then allowed to freeze for the remainder of the 7-day period, show 55, 66, 71 and 75 per cent of the potential 7-day strength.

In Fig. 3 the specimens received the same early curing treatment as those in Figs. 1 and 2—but at the age of 7-days were returned to the warm air of the laboratory where they remained until the 14-day period, at which time they were broken. The direction taken by Curves L, M, N and O seems to indicate that at some period beyond the age of 14-days these curves would join the potential strength curve. It is regretted that this series does not include definite data on Curves L, M, N and O at the later ages, however, this is one of the series which will be developed further in the coming winter.

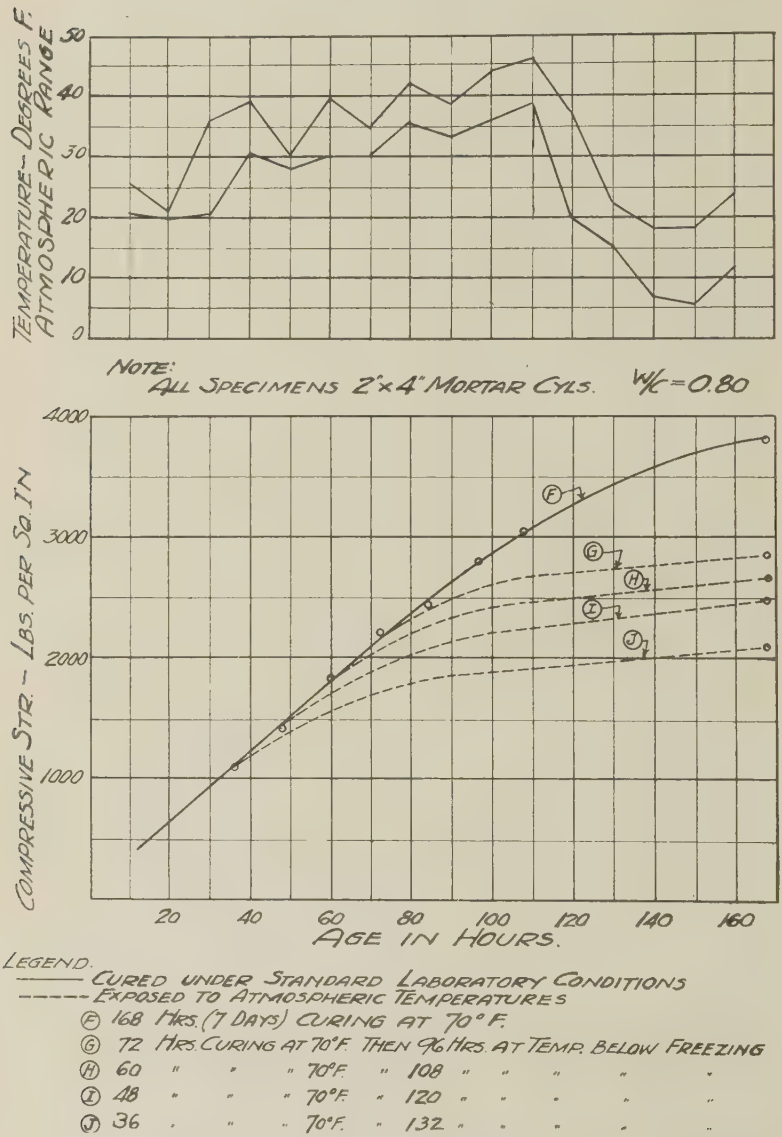
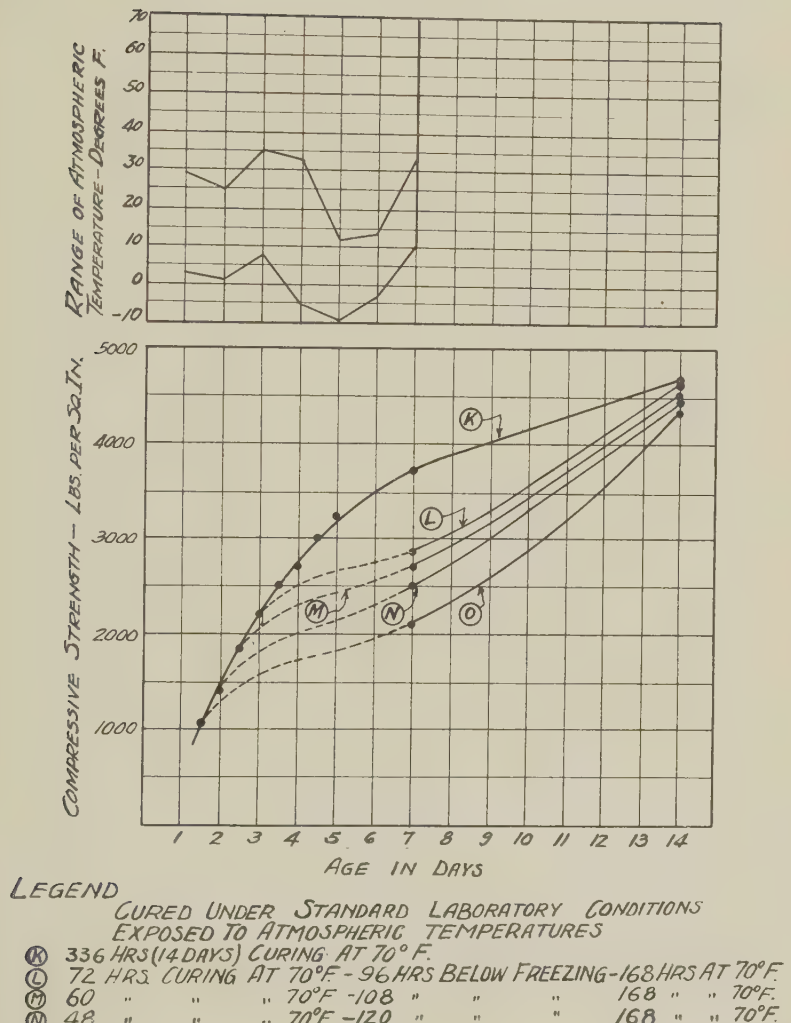


FIG. 2—EFFECT OF EARLY CURING AT LOW TEMPERATURES ON 7-DAY STRENGTH



LEGEND

CURED UNDER STANDARD LABORATORY CONDITIONS EXPOSED TO ATMOSPHERIC TEMPERATURES

(K)	336 HRS (14 DAYS) CURING AT 70° F.								
(L)	72 HRS. CURING AT 70°F - 96 HRS BELOW FREEZING - 168 HRS AT 70°F.								
(M)	60	"	"	"	70°F -108	"	"	"	168 " " 70°F.
(N)	48	"	"	"	70°F -120	"	"	"	168 " " 70°F.
(O)	36	"	"	"	70°F -132	"	"	"	168 " " 70°F.

FIG. 3—STRENGTH GAIN WHEN CURED IN WARM AIR AFTER INITIAL CURING AT LOW TEMPERATURES

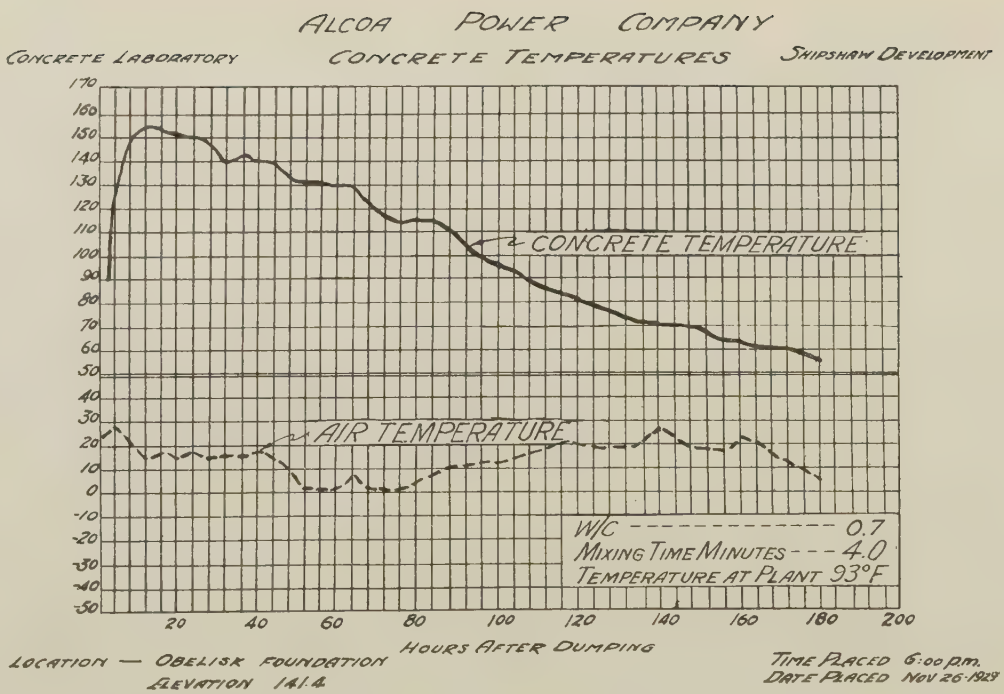


FIG. 4—TEMPERATURE RANGE OF MASS CONCRETE

During the time that the tests referred to above were being made, an arbitrary curing period of 72-hours was specified for concrete placed in all structures on the Chute a Caron project. By this is meant that adequate heat should be supplied, and proper covering provided to retain that heat, so that the temperature of the concrete 1-in. from the form faces would not fall below 60° for 72-hours after placing.

In general, this specification gave excellent results. In certain sections of the heavy mass work it was found that the form covering could be removed and the steam heat discontinued after 36-hours and the temperature of concrete near the exposed surfaces would still be 50 degrees or above at 72-hours after placing. On other smaller sections of the Power House work it was found necessary to keep the covering in place and the steam heat circulating for the full 72-hour period.

The chart, Fig. 4, gives a good idea of the temperature range on mass work. It will be noted that the concrete left the mixing plant at 93° F. The temperature began to rise immediately after placing and continued to do so until it had reached about 155° at 12-hours. It then decreased gradually until it reached 55° at 180-hours. In this instance the protective covering was kept in place for 60-hours.

It is not a difficult matter to work out a satisfactory type of form covering. Charles S. Hill, in his book "Winter Construction Methods" gives several excellent examples. The methods used at Chute a Caron Dam are shown in Figs. 5 and 6. It will be noted that practically the same type of covering is used on the small sections (Fig. 5) as on the heavy mass work (Fig. 6). The form sheeting is made of 2-in. lumber and this has considerable insulating value in itself. The covering for vertical faces consists of 4 by 10-ft. panels, burlap and tar-paper covered. These may be nailed to the main form walers, or placed about 18-in. back from the main form to provide a working space for pipefitters. A 1-in. steam line is run around the form at the bottom and at the center of each lift. The opening between the two forms is closed at the bottom to prevent upward draft. The top is covered by tarpaulins. Over large forms the tarpaulins are supported on

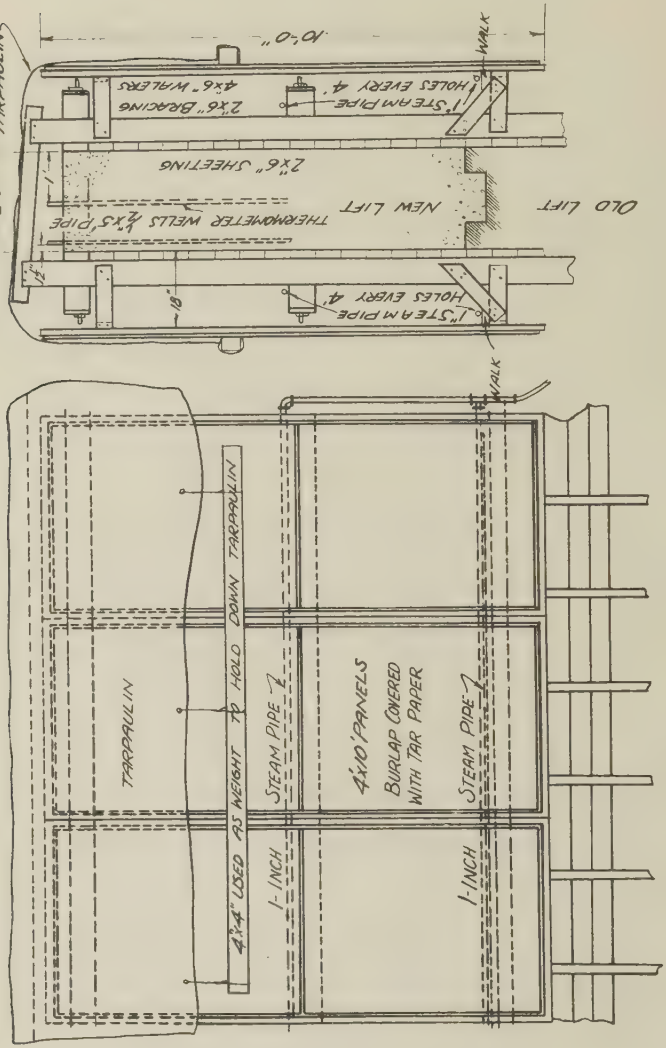


FIG. 5—METHOD OF PROTECTING SMALL SECTIONS OF CONCRETE

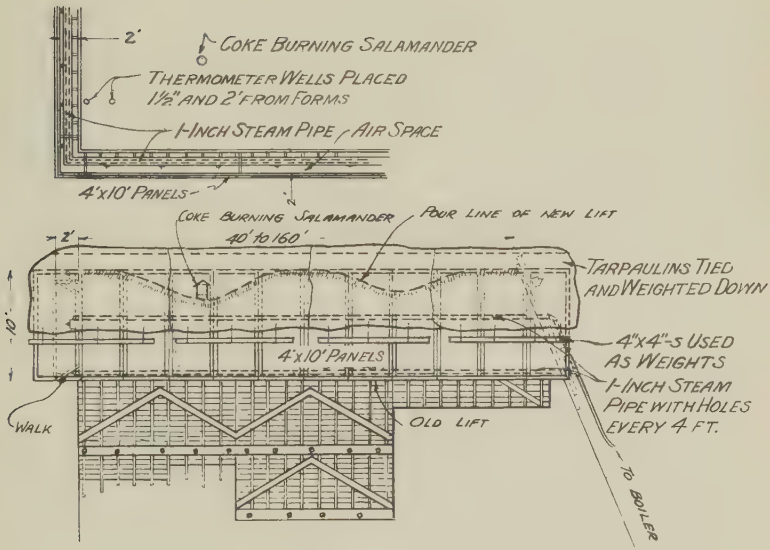


FIG. 6—METHOD OF PROTECTING HEAVY MASS WORK

light frame-work, and coke-burning salamanders are placed in the shear valleys, as shown in Fig. 6.

It is recognized that live steam, due to the moisture supplied, provides better curing protection than dry coils—however, it was found necessary to arrange the piping so that both types could be used on this work. If live steam is used while the placing is in progress, clouds of steam prevent the crews from seeing and the work becomes difficult and dangerous. Valves are placed at intervals in the coils so that dry heat may be supplied during the time the form is being filled, and then the valves are opened and live steam supplied when the form is finished and covered. Thermometer wells made of ½-in. water pipe, from 3 to 8-ft. long, are placed in the lift while the concrete is soft. These are normally located in three positions in the form; one at the up-stream face, one near the center and one at the down-stream face. Temperature readings are taken in these wells during the curing period.

When the air temperature is below the freezing point, necessary arrangements are made at the mixing plant to produce concrete having a temperature of from 70° F. to 100° F. It was found that concrete heated to above 100° was in danger of taking a flash-set. On two occasions when the mixture was discharged at 110° the concrete was set up in the buckets before it could be delivered to the forms (a matter of 25 or 30 minutes). This may have been caused by certain characteristics of the particular brand of cement used, or, it may have been caused by the manner in which the materials were introduced into the mixer drum. It has been suggested that if the cement came into immediate contact with mixing water at a temperature of from 180° to 200°, the chance of a flash-set would be much greater than when the mixing water had first been cooled to some extent by contact with the sand and stone. It is the opinion of the authors that there is no advantage in heating concrete higher than to 100°. This temperature offers ample protection for any ordinary transportation and placing system and has been found to be quite satisfactory where air temperatures are as low as 40° below zero.

Mr. Johnson has stated that concrete should be placed in the form at not less than 70° and not more than 140°. The lower limit would be quite satisfactory for reinforced frame-work, but it should be pointed out that temperatures from 10° to 15° lower than this can be used with safety on mass work where the air temperature is anywhere above zero. This is assuming that proper protection at the form is provided. His upper limit of 140° is open to question and there seems to be a great difference of opinion on the subject. The authors' views on this point have been stated in the preceding paragraph.

The matter of heating aggregates is not a serious problem where proper facilities are provided. It is probably a waste of time and money to try to heat the coarse aggregate at all. If enough stone or gravel is to be used to feed even a 3-bag mixer continuously the time and money required to heat this material is out of all proportion to the benefits obtained. Instances are very rare where more than a small portion of the stone particles are ice-coated and the heat of the mixing water alone is enough to thaw any such ice-coating before the concrete leaves the mixer.

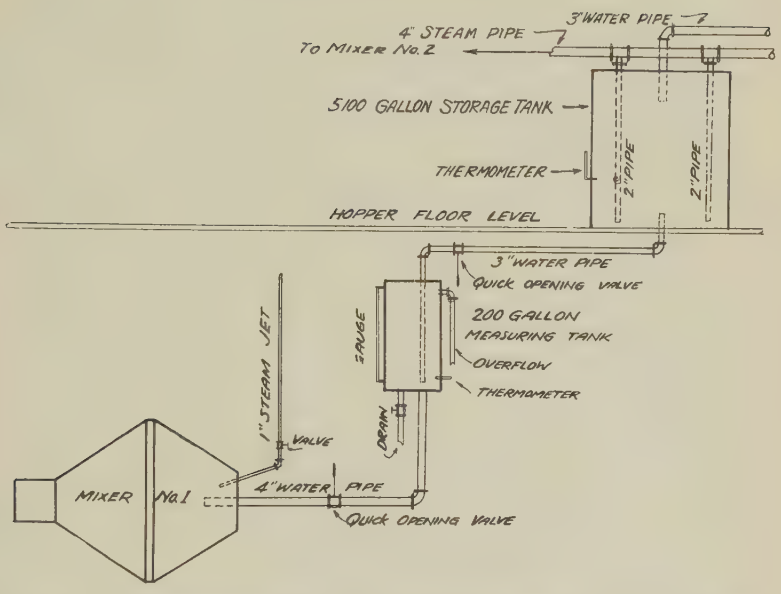


FIG. 7—ARRANGEMENT FOR SUPPLYING STEAM AND HOT WATER TO MIXERS

By using live steam it is a simple matter to heat sand to almost any temperature from 60° to 150°. On our work the sand temperature usually runs between 70° and 80°—with the mixing water at from 180° to 200°. This combination, together with a 1-in. steam jet in the mixer, will produce concrete at from 70° to 100°.

Mr. Johnson recommends that the sand and stone be heated when the air temperature is 35° F. and lower. This precaution would certainly provide a comfortable margin of safety. However, the authors are of the opinion that in some cases it would add an unnecessary item to the cost of winter construction. Where the efficiency of the form protection scheme is questionable it might be a worthwhile precaution to heat the aggregates as Mr. Johnson suggests. It has been the experience of the authors that heating the mixing water only, will provide ample protection until the air temperature gets down to about 15° above zero.

The method used at Chute a Caron for supplying hot water and steam to the mixing plant may be best understood by study-

ing the diagram, Fig. 7. The 4-in. pipe-line, which supplies the steam for heating water in the two 5100-gal. storage tanks, is taken from two 80-h. p. boilers. These boilers also furnish steam for the 1-in. jets in the mixer drums and for the various jets in the sand stock-pile.

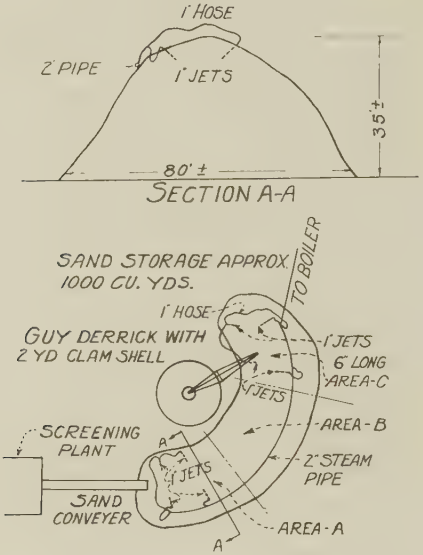


FIG. 8—METHOD OF HEATING SAND IN STORAGE PILE

The sand in the storage pile is heated by several 1-in. steam jets which are moved about as conditions require. The storage pile is divided roughly into three areas, two of which are being steamed while sand is being used from the third. A sketch showing the sand heating arrangement is shown in Fig. 8.

If conditions do not favor the storage pile arrangement, and it is found more convenient to deliver sand directly from the pit to mixing plant bins, a very satisfactory method of heating is to provide steam jets in the dump cars, as shown in Fig. 9. Eight 1-in. steam jets placed in a 30-yd. car will thaw all frozen lumps and heat the sand to about 100° F. in about 3-hours. If the sand

is to be screened before using, an added benefit of heating in the cars lies in the fact that the hot sand prevents the conveyor belts and screens from freezing. The maximum loss of heat caused by handling sand from cars, through the screening plant and into the mixer bins, was found to be about 20 degrees.

While the heating and protection arrangements outlined above have given very satisfactory results at Chute a Caron Dam, there

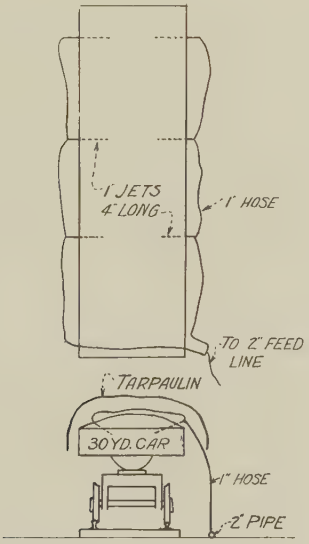


FIG. 9—METHOD OF HEATING SAND IN DUMP CARS

are doubtless many other methods in use which are just as good, or better. It would be a fortunate circumstance if Mr. Johnson's committee could be successful in getting all engineers who are familiar with cold weather concreting work to record their experience in order that the most efficient methods could be described in detail and published in his committee's final report. If "Recommended Practice for Winter Concreting" is to be written and sponsored by the Institute, the economic phase should be given very careful consideration. While the ultimate

aim of such a specification should of course be to produce concrete of the quality prescribed by the design, the requirements of the specification should be such as to avoid unnecessary expense. A constructor should not be required to keep a section of his concrete work covered and heated for 10 days if 3-days will suffice—neither should he be required to go to the expense of heating aggregates before conditions warrant such action.

Discussion of Report of Committee 403

"The Treatment of Monolithic Concrete Surfaces"*

BY H. M. HADLEY†

The report of Committee 403, Monolithic Concrete Surfaces, by N. C. Johnson, is good reading. Definite ideas are definitely expressed. There is nothing vague or nebulous about Mr. Johnson's statements, and whether in agreement or disagreement with them, one finds them refreshingly explicit and to the point.

Mr. Johnson looks for major developments in concrete as a surfacing material of artistic worth by removal of "the objectionable and harmful cement form skin" and exposure of the aggregate, either without further treatment or as a clean sound base for veneers. He properly enumerates the several defects and disadvantages which may singly, or severally, or—also be it said—may not at all occur with form-cast surfaces. "The constant and resistless and never-ending action and reaction of this surface excess of cement with atmospheric moisture" does indeed lead to "unfortunate volumetric and appearance changes" sometimes and under certain conditions, but such changes are not inevitable. In fact, the writer's observations, not wholly confined to "balmy climates of almost constant humidity," would lead him to think that concrete surfaces, untouched and direct from the form boards, withstood the attrition of weathering processes in a highly satisfactory manner wherever the concrete itself was of sound, good quality. Where the quality was not good, that fact also was revealed. It is impossible to run a bluff on nature. But

*By N. C. Johnson, author-chairman; Journal, May 1930—A. C. I. *Proceedings*, Vol. 26, p. 717.

†Regional Structural Engineer, Pacific Coast Region, Portland Cement Association, Seattle, Washington



FIG. 1—LIFE SCIENCE BUILDING, UNIVERSITY OF CALIFORNIA, BERKELEY. GEORGE W. KELHAM, ARCHITECT. ENTIRE STRUCTURE OF MONOLITHIC CONCRETE, INCLUDING ORNAMENT

the opinion is submitted that concrete of good structure and quality possesses a form-cast surface requiring no additional treatment to render it enduring and permanent, and, in this respect at least, entirely satisfactory.

It is because the great and truly impressive growth of architectural concrete on the Pacific Coast has proceeded almost wholly through the development of the form-cast surface that fuller mention of such treatments should be made. These form-cast surfaces have been wash-coated, acid-stained, or painted for color as the case may be, or may have been left in natural color, but they are all characterized by the distinct preservation of the form marks and form surfaces. Such architectural designs emphasize *concrete*, in contrast with designs in which veneers,

either of mortar or masonry, conceal it. They present construction problems, some of a general and some of a peculiar nature, but they also offer opportunities for the exercise of freedom in architectural design which is delightful and the beauties of whose results cannot but command admiration and respect.*

He was a brave man who ate the first oyster, it is said. Quite his peer is that nameless architect who first dared to leave concrete frankly exposed, with the form-marks showing and the grain of the wood revealed and imperfections of various minor kinds all plain to view. Here a board bulged slightly; there was a knot; over there a nail bent for some cause or other and a carpenter swore. But the nameless architect who knew that according to all the conventions it ought to be plastered or

*See "Architectural Concrete—Forms, Molds and Surfaces" by William C. Wagner, in the pages of this JOURNAL issue.—EDITOR.

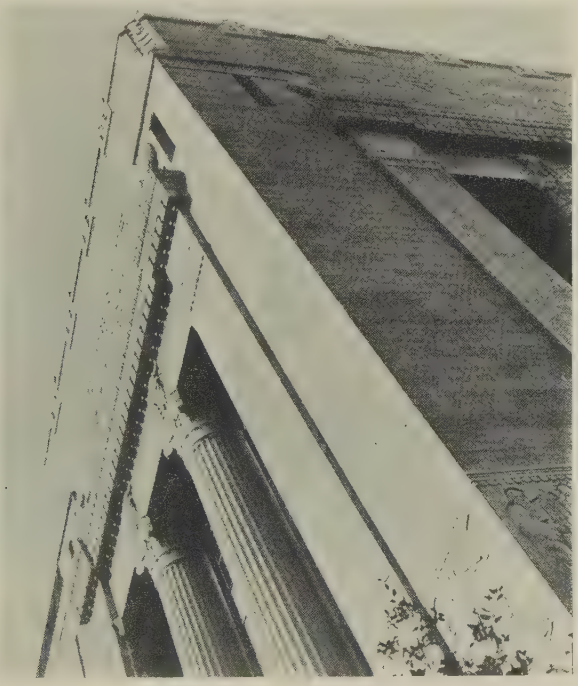


FIG. 2—DETAIL OF LIFE SCIENCE BUILDING, SHOWN IN FIG. 1



FIG. 3—ENTRANCE HAWTHORNE SCHOOL, BEVERLY HILLS, CALIFORNIA. R. C. FLEWELLING, ARCHITECT

veneered, also knew in his own mind that this surface with natural random variation of line and texture, just as it came from the forms, was rich in interest and was as good as any veneer could possibly make it. Why not, when the forms and molds are as characteristic of concrete, are as intimately and as essentially part of the construction, as is the concrete itself? It is basically good in everything, architecture included, to be honest. So since the cost of the veneer he had planned on could be used to take care of one or two troublesome "extras" that had developed on the job, and since he had pretty good concrete in the walls anyway, our nameless architect nerved himself and decided to leave it exposed with only a wash coat to give it color. And he got away with it, and later found that he liked it more and more, and found that after the first shock, other people did, too; and so on.

Probably such are the details of this first and unknown case. Be they what they may, the later uses are known and stand in evidence, and of their beauty and charm there can be no question. They are accomplished facts. See Figs. 1, 2 and 3. For buildings of more formal character to which the ordinary textures of wood are less well adapted, extremely true plane surfaces are obtainable by lining the wooden forms with sheets of pressed wood fiber. Jointing and rustication of such surfaces is obtained by nailing beveled strips over the fiber board lining. Again, various formal surfaces of large area may be obtained with wooden forms, milled to desired pattern, shellacked and oiled. For more elaborate decorative effects, waste molds of plaster, or wooden molds, are built in with the wooden forms and offer wide scope for variety in design. Accompanying views are representative of the results being obtained with treatments of this type.

Color effects may be produced in this type of construction where other than the natural tone of the cement is desired either by wash coats or by the integral admixture of color in the cement. The Wilshire Professional Building derives its white color from a paint coat. The exposed concrete of the Richfield Oil Building—approximately half of the entire exterior—was made to conform with the black terra-cotta of the main facades by covering it with a China wood-oil carrying a carbon pigment.



FIG. 4—BANK OF HOLLYWOOD BUILDING. TWO LOWER STORIES ON STREET FRONT ARE TERRA COTTA; ABOVE, ALL CONCRETE WITH SURFACES AS LEFT BY PRESSED FIBRE FORM LINING AND ACID STAINING BY LAEMMENS PROCESS

It cannot be too strongly emphasized that with designs contemplating exposed concrete exteriors, the concrete, just as the name indicates, is really and actually to be exposed in the finished structure and will be in the plain sight and view of everyone. This means that comfortable, easy-going, good-enough concrete, whose defects can so easily be concealed by veneers, is eliminated at the outset. It simply cannot be tolerated because it would be an

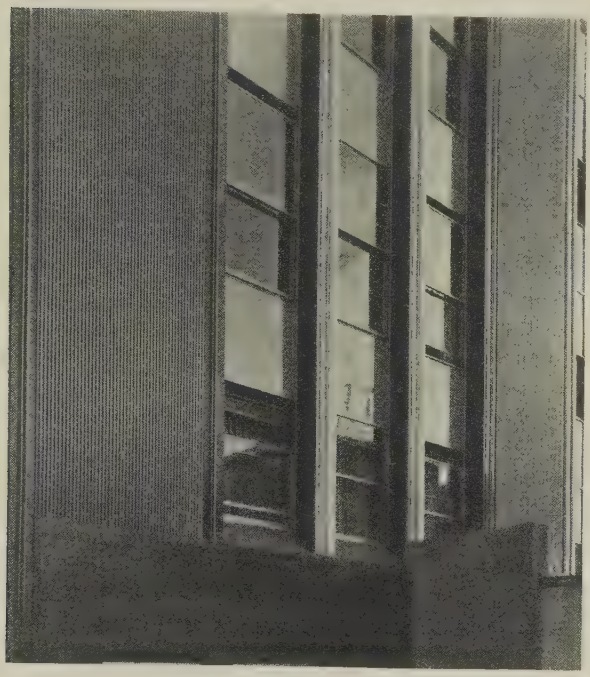


FIG. 5—DETAIL OF CONCRETE PORTION OF RICHFIELD OIL BUILDING, LOS ANGELES. TERRA COTTA ON STREET FRONTS; CONCRETE ON PROPERTY LINE SIDES; WOOD FORMS WERE MILLED AND FLUTED TO CONFORM WITH TERRA COTTA

eyesore to all beholders and a continued standing indictment of the contractor who built it and of the architect who permitted it to be built. It would be better not to attempt exposed concrete than to fail at it. Yet successful results can be obtained readily. What is required is a concrete surface of uniform quality, free from gravelly, honeycombed blotches, segregation of materials, ragged and irregular pour lines, laitance or sawdust in construction joints, badly sprung forms—in fact a concrete free from major and serious defects. There must be obtained as near to a full 100 per cent extent as possible that same excellent good quality of concrete surface which even a poor job will show over

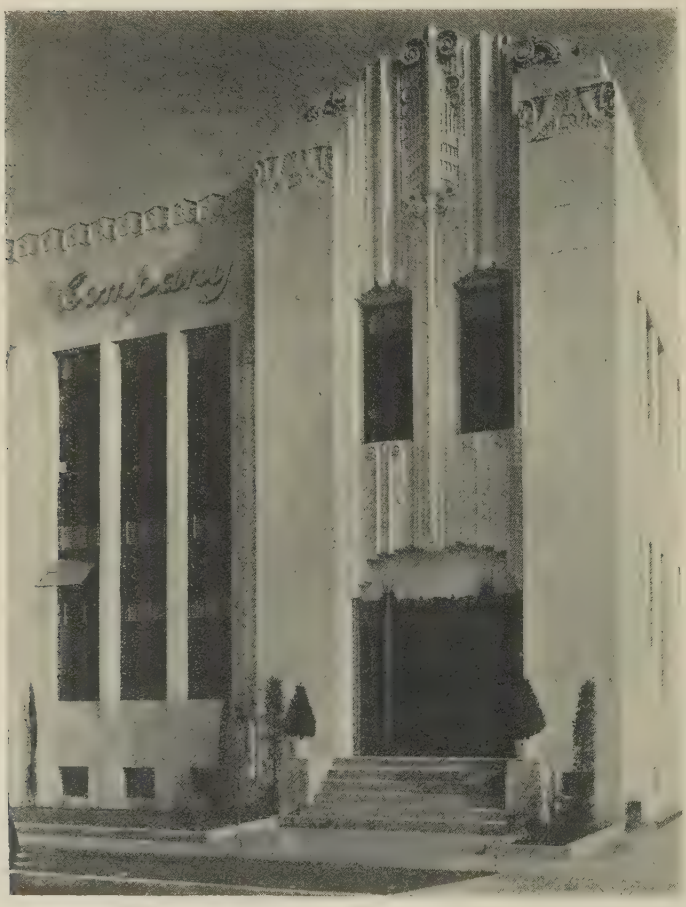


FIG. 6—HOFFMAN CANDY CO. BUILDING, LOS ANGELES. PLASTER WASTE MOLDS USED TO PRODUCE ORNAMENTATION

most of its area. A plastic workable mixture of minimum water content has to be placed to fill completely the forms which have been built to receive it and in such a manner as to result in a homogeneous, uniform concrete. Merely superficial blemishes—air pockets, water voids, exposure of fine honeycombing of no depth —are not at issue. They give variety and attractiveness and interest and prevent the monotony of uniform perfection. But

the question of uniform, good concrete is of issue and has to be met.

The architectural uses of concrete are to be welcomed by everyone concerned with improving the quality of concrete. To bring concrete into the focus of attention, to make it visible, subject to the critical examination of all eyes, will almost automatically eliminate careless slipshod work. When slipshod work can not be hidden but has to stand in plain view, nobody wants it and everybody disclaims it. It becomes inexcusable, intolerable. To subject concrete to such a searching scrutiny and inspection as it must of necessity receive when used architecturally will have a very wholesome effect upon its quality.

BY B. STUART MCKENZIE*

In my construction experience I became acquainted with two methods of finishing concrete surfaces which have not been mentioned. Both methods proved most satisfactory in service.

The first consisted in the application of a cement grout wash rubbed in by means of a canvas sack, usually an old cement bag, immediately after the removal of the forms. The result was much more satisfactory than that obtained by the use of a whitewash brush, which is the usual custom.

The other method consisted in the use of a hand block made of cement mortar. The surface to be treated was thoroughly wet with a whitewash brush and then rubbed with the cement block. This served to grind off the surface imperfections and also to fill voids and produced a surface which was practically perfect.

It may be objected that these methods, particularly the latter method, are too expensive from an operating standpoint but in my own experience the results obtained were well worth the cost.

AUTHOR'S CLOSURE BY NATHAN C. JOHNSON

Discussion of any subject is always most interesting. The discussion by Mr. McKenzie and Mr. Hadley are welcome sidelights on this subject of surfacing, which has too long been neglected.

The writer intended, in speaking of "Wash and Float Finishes," to include by inference the methods mentioned by Mr. McKenzie.

*Secretary, Canadian Engineering Standards Association.

Beyond this acknowledgment, there seems no need to amend the text of the report as it is.

The comment of Mr. Hadley is particularly forceful in one sentence.

This sentence is: "It is impossible to run a bluff on Nature."

This sentence, perforce, runs through all of Mr. Hadley's comment. Each paragraph of that comment should have, as an opening sentence, this deep truth, that: "It is impossible to run a bluff on Nature."

Mr. Hadley brings California to us once again, but all the concrete in the world is not in Los Angeles. For every photo he will show in illustration of his comment, any camera in any other clime or city could bring other photos, sometimes unbeautiful in the extreme, proving by concrete buildings elsewhere, that "It is impossible to run a bluff on Nature."

It is recognized that Mr. Hadley's comment is familiar propaganda. But is it helpful propaganda? Unless a statement of position tends to advance or improve an art, over what has been done before in that art, it may well prove to be harmful and delusive.

In Egypt, thousands and thousands of years ago, long before portland cement was thought of, the artisans of that country colored, stained, painted and otherwise over-treated mud surfaces (form surfaces, if the term is preferred) because they were unlovely as they were and because it was even then recognized that they had to be and should be covered up to make them "good enough," even for those days.

And they succeeded then, because the climate of Egypt is so ridiculously dry that even unbaked mud bricks of those past ages have endured to the present day.

In Los Angeles, where humidity is so constant, one can and may cover up unlovely form skins with stains, varnishes, china-wood paints and other means as mentioned. But to infer or to state that in other and unlike parts of the world these things may be successfully done, is no kindness to the art of using concrete, for "It is impossible to run a bluff on Nature."

In detail, one cannot but wonder if Mr. Hadley ever attempted to make or to cast an intricate form, or to line a form with pressed wood fiber and so on. Some of us have.

It is also a grave mistake to talk, except perhaps in California, of painting concrete with china-wood or linseed paints, without special pre-treatment.

Other comments on the factual and economic aspects of Mr. Hadley's views might properly be made and to the advantage of the industry, but if it be remembered in reading them, that it is, in very truth, "impossible to run a bluff on Nature," much time will be saved.

The Committee report as it stands, it is true, mentions things that our grandfathers did not know about, although they knew all about form-mark surfaces. But in no industry that we know of is enterprise and improvement considered a crime, except in the minds of the few and even with them, it is a crime for a brief time only.

As Dr. Aggasiz so aptly phrased it many years ago:

> Every advance passes through three stages: First, men say it is not true. Then, they declare it is hostile to accepted beliefs. Finally, they say that it is right and is true and that everyone had always known it.

The Committee report as it stands does not seem to need correction or modification.

Strength and Shrinkage of Mortars Made with Blends of Portland Cement and Pozzuolanic Materials

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A POZZUOLANIC material has been defined[1] as "a substance, while not necessarily cementitious by itself, possesses constituents which will combine with hydrated lime at ordinary temperatures in the presence of moisture to form stable insoluble compounds of cementitious value. When added to lime mortar in addition to or in partial substitution for the sand, it will impart hydraulic properties and greater strength. When added to concretes and mortars made with portland and similar cements by the consequent removal of the free lime, it will impart to the material an increased degree of resistance to various agencies which are normally liable to cause disintegration."

If it be accepted[2] that the compounds formed by a pozzuolana and the lime hydroxide liberated during the hydration of Portland cement are of cementitious value, then the use of pozzuolanas with Portland cement will affect the water-cement ratio strength relations.

It is claimed[3] also that pozzuolanic admixtures reduce defects caused by shrinkage, particularly crazing and checking.

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[1] "*Lime and Lime Mortars*". A. S. Cowper, Special Report No. 9, Department of Scientific and Industrial Research. London.

[2] "Tufa Cement, as Manufactured and Used on the Los Angeles Aqueduct"—J. P. Lippincott, *Transactions of American Society of Civil Engineers*. Vol. 76, 1913, page 520. This paper and discussion containing much data from tests made during the construction of the Los Angeles Aqueduct and elsewhere is to the authors' knowledge, the most complete treatment of blended cements as used in America. Mr. Lippincott's conclusions are as follows:

[3] An American Pozzuolana and Its Effect on Portland cement Concrete. Dr. E. Lee Heidenreich, *Rock Products*. Sept. 29 and Oct. 27, 1928.

1st.—The tufa, when finely ground with cement and used in concrete, combines both chemically and mechanically. Blends of 50%, when mixed with sand, give greater tensile strength after 10 days than straight cement mixed with the same proportion of sand. The leaner the mixture, the greater the relative superiority of the tufa cement. In compression, the tufa cement concrete is less strong (20%) in rich mixtures (1:2:4), and as strong in leaner mixtures (1:3:6). (footnote cont'd p. 318)

The tests for which the data are presented herewith were undertaken to determine the effect of four pozzuolanic materials on the strength and shrinkage of mortars. Compression tests were made on 218-2x4 in. cylinders and shrinkage measurements taken on 27-2x2x10 in. beams.

MATERIALS

Cement. Two brands of portland cement were used. These are designated by the letters N and A. The N brand was used for the majority of the tests. Both cements complied with the requirements of the A. S. T. M. specifications.

Fine Aggregate. The fine aggregate was a local concrete sand graded from the No. 4 sieve down with a fineness modulus of 2.6. It was used in a bin-dry condition and had a net absorption of 0.6 per cent. Care was taken to maintain a uniform grading for all mixes.

Water. The mixing water was from the Minneapolis City supply and was at 70° F. when used.

Pozzuolanic Materials.

F—a volcanic ash obtained from a deposit near Fresno, Calif.

P—a volcanic ash sold commercially as a concrete admixture.

S—Blast furnace slag used as a raw material in the manufacture of portland cement.

H—Burnt shale sold commercially for concrete aggregate.

F and P were used as received. S and H were ground in a ball mill. No measure of the fineness of the pozzuolanas was obtained other than that all passed the No. 200 sieve, which is of little significance since pozzuolanic action results from the very fine particles.

The pozzuolanas were substituted for cement by weight so that the total weight of cement or cement plus pozzuolana was constant for all mixes. The cement and pozzuolana were blended to a uniform color before mixing the mortars.

2nd.—Tufa cements, in tension, of blends from 30 per cent to 80 per cent show a continued growth in strength with age, as far as tested, up to 5 years, and in this respect are superior to straight cements, which usually show declining strengths.

3rd.—The tufa concretes must be handled with greater care with reference to both cold and drying, and forms should be left in place about one-third longer. In massive work this feature is negligible.

4th.—From the fact that the tufa cement is more finely ground and, in part, combines mechanically with other aggregates, carrying the graduation of fineness one step farther, it makes a denser and more impervious concrete.

5th.—Where cements are high priced, a substantial economy may be effected if deposits of tufa are available. These conditions occur in portions of Western America. The milling cost of producing the extra barrel of tufa cement in small plants should not exceed 75 cents.

Pozzuolanic Activity

The relative activity of the four pozzuolanas was determined by a method used by the Building Research Board.[1] This consisted of adding 0.6 g. of pozzuolana to 0.4 g. of slaked lime in a test tube, covering with pure water and shaking every twelve hours.

The interaction between the solution of lime in water and the pozzuolana will give rise to hydrated calcium alumino-silicates of complex or indefinite chemical structure; these hydrates will be formed in a colloidal condition, and are much more bulky than the pozzuolana or lime, with the result that after seven days the tube will contain an increased volume of solid matter as measured by the height it extends up the tube, also the rate of settlement, owing to its flocculent nature will be slower; the activity of the pozzuolana can, therefore, be judged by inspecting the tubes at different times after shaking. In comparison, the volume of the solid matter immediately after mixing will be much smaller and it will settle immediately.[1]

The per cent increases in volume following a 30 minute settling period are shown in Table 1. Also see Fig. 1.

TABLE 1—PER CENT INCREASES IN VOLUME RESULTING FROM POZZUOLANIC ACTIVITY

Age Days	Pozzuolana			
	F	P	S	H
2	8	64	16	52
3	46	79	236	73
4	92	89	245	83
5	123	97	245	94
6	143	100	247	115
7	146	100	254	135

Mixing. From each batch consisting of 1500 grams of sand and 500 grams of cement, or cement plus pozzuolana, were made three 2x4 in. cylinders or one beam. The method of mixing the mortars was that specified by the Standard Specification for Portland cement. Four sets of cylinders and two sets of beams were made, each set being molded on separate days.

Consistency. All mixes were brought to a "just plastic" condition. The quantity of water required for each mix was determined from preliminary trial batches and checked on the mixes used in making the specimens. In the trial batches, water

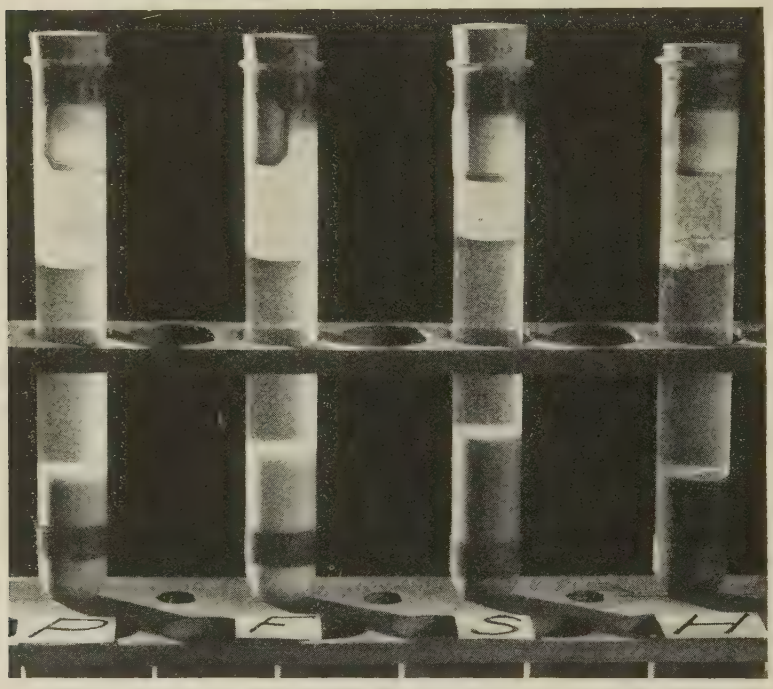


FIG. 1—TEST OF POZZUOLANIC ACTIVITY

The top edges of the black tapes at the bottoms of the test tubes are at the original levels of the mixtures of pozzuolanic materials and lime.

was added in increments until the mix changed from a more or less granular to a plastic, cohesive state. This change is well defined for most mortars. The quantities of water determined by independent trials checked closely. The mixes when "just plastic" were considered to be of equal consistency.

Curing. All specimens were placed in a moist closet immediately after molding and stripped at 20 to 22 hrs. All compression specimens were then placed in water storage at 70°. The subsequent curing is recorded in Table 2. The curing conditions for the shrinkage specimens following stripping is given in Table 4.

Testing. All specimens were tested in the moist condition, the air-cured specimens being submerged in water for 24 hours im-

TABLE 2—COMPRESSIVE TESTS.

Specimens—2 x 4 in. cylinders.
Mix—1:3 by weight.
Pozzuolanas substituted for cement and blended with it before mixing mortars.
Water used in calculating the ratios of water to cement and water to cement plus pozzuolana corrected for the absorption of the sand only. The absorption of the pozzuolana is not considered.
All strengths are an average of four specimens made on different days, except as noted.
All specimens submerged 24 hours prior to testing.

Pozzuolana	Water-Cement Ratio (Vols.)	Ratio—Water to Cement Plus Pozzuolana (Wt.)	Abrams Water Factor for Admixture	Compressive Strength Lb. per Sq. In.			Strength Ratios Based on Plain Mixes			Curing
				7 Day	28 Day	90 Day	7 Day	28 Day	90 Day	
N Cement—10 Per Cent Substitution by Weight.										
None	0.66	0.437	10	2620[2]	4060	5480	100	100	100	1 day moist
F	0.76	0.457	10	2220	3880	4585	85	96	84	room
P	0.82	0.492	45	1730	3565	4840	66	88	89	Remainder
S	0.75	0.452	5	2030	4330	5600	77	107	102	under water
H	0.77	0.462	15	2215	4230	6045	85	104	110	at 70°.
A Cement—10 Per Cent Substitution by Weight.										
None	0.63	0.417	10	3790	5640	6560	100	100	100	
F	0.74	0.442	15	2820	4600	5020	74	82	77	
P	0.79	0.472	45	2805	4320	5610	74	77	85	ditto
S	0.73	0.437	10	3075	5010	5275	81	89	80	
H	0.74	0.442	15	3210	5010	6790	85	89	103	
N Cement—30 Per Cent Substitution by Weight.										
None	0.66	0.437	10	2620[2]	4060	5480	100	100	100	
F	1.05	0.482	5	1230	2610	4430	47	64	81	
P	1.25	0.592	41	925	2100	4615	35	52	84	ditto
S	0.98	0.457	—3	1830	4230	6405	70	104	117	
H	0.93	0.432	—8	1785	3490	6390	68	86	116	
None				2620[2]	3010	3030	100	100	100	1 day moist
F				1265	1740	1940	48	58	64	room, 6 days un-
P	Ditto			840	1260	1320	32	42	44	der water at 70°
S				1715	2670	2870	65	89	95	remainder in air
H				1745	2570	2990	67	87	99	of Laboratory.[1]

[1]Average of 3 specimens.
[2]Average of 8 specimens.

mediately prior to testing. One specimen from each batch was tested at each of the ages of 7, 28 and 90 days.

Shrinkage Measurements. The shrinkage measurements were taken with a Berry strain gauge on an 8-in. gauge length.

Initial readings were taken on the shrinkage specimens at 20 hr. for the first or "a" set and at 22 hr. on the second or "b" set. Drilled and countersunk brass plugs were used for gauge points. Three standards and all specimens of one set were read with one setting of the dial. Two such series of readings were taken each time, the dial being shifted between series. Third readings were taken when necessary. The relative movements were then calculated from the averages of those differences not varying by more than four "points" which corresponds to a movement of 0.001 per cent.

The per cent linear changes relative to the 1 day (20 and 22 hr.) lengths and the differences between the "a" and "b" specimens are given in Table 3 for 5 ages. The differences shown vary from a maximum of 0.0128 to a minimum of 0 and have an average value of 0.0033 per cent relative movement. This agreement is considered satisfactory, particularly in view of the fact that the "a" and "b" sets were made one week apart.

DISCUSSION OF RESULTS

In Table 2 the quantity of effective water is expressed both as a ratio of the quantity of portland cement by bulk volume, and as a ratio by weight of the blended cement. The quantity of effective water* used in each batch is easily obtained by multiplying the latter ratio by 500. With one exception (the mix containing 30 per cent substitution of H) more water was required for the blended mixes than for the plain mixes. Except for the mixes containing P, the additional quantities of water required were small, but since the quantities of portland cement were reduced, the differences in water-cement ratio are considerable.

Abram's "water factor"[4] defined as "the quantity of mixing water required by the addition of a unit weight of any constituent in order to maintain a given workability," is given for each

*The term effective water is used here to mean the quantity of added water corrected for the absorption of the aggregate, no allowance being made for absorption by the pozzuolanas.

[4]Duff A. Abrams, "Tests of Powdered Admixtures in Concrete." Proc. A. S. T. M., Vol. 29, 1929, Part II, p. 627.

material in the fourth column of Table 2. These were calculated on an assumed water-factor for cement of 10 per cent. Taking into account the fact that small changes in the total quantity of water result in very large changes in the water factor, the agreement between the values obtained for the blends of the two cements and the two percentages of each pozzuolana is good.

The authors believe that the accuracy of the method used in determining the quantity of water may be judged by comparison of the cement factors for the same pozzuolana obtained from different mixes. The ratios of water to cement plus pozzuolana are given to three decimal places for convenience in checking since the use of these figures when rounded to two places would give cement factors appreciably different from those shown.

Compressive Strength

The relative strengths, plotted from the data of Table 2 are shown in Fig. 2. All four blends show a considerable loss in strength at 7 days for both conditions of curing. In general, the compressive strengths of the blended mortars increase relative to those of the plain mortars with increase in age. Materials S and P, showing the greatest and least pozzuolanic activity respectively, also had the greatest and least relative strengths.

Moist cured specimens. With the N cement, the use of both 10 and 30 per cent substitutions of S and H gave relative strengths at 90 days greater than 100. With both N and A cements the 10 per cent substitution of F and S resulted in some retrogression at 90 days from the 28 day relative strengths. The average relative strength of the 10 per cent blended specimens made with N cement were about equal at 7 days and considerably higher at 28 days than that of those made with A cement.

In general, there is considerable difference in the data given by the two brands. Assuming that there is chemical action due to the pozzuolana, it is natural to expect different effects with different brands of portland cement, as has been found to be the case with other chemically active materials such as calcium chloride.[5]

Air cured specimens. These specimens were moist cured for the first seven days and then left in air. While the small specimens

[5] "Calcium Chloride as an Admixture in Concrete." Bull. 13, Structural Materials Research Laboratory. Lewis Institute. See Fig. 11.

TABLE 3—PER CENT CHANGE IN LENGTH AND VARIATION BETWEEN "a" AND "b" SPECIMENS

Age of Specimen Days	7	142	156	201	339
Specimen	N Cement—10 Per Cent Substitution				
Na	—0.0025	+0.0982	+0.0627	+0.1140	+0.1285
Nb	—0.0057	+0.0945	+0.0615	+0.1120	
diff.	0.0032	0.0037	0.0012	0.0020	
N1Fa	—0.0057	+0.1012	+0.0662	+0.1140	+0.1280
N1Fb	—0.0060	+0.1055	+0.0667	+0.1182	+0.1335
diff.	0.0003	0.0043	0.0005	0.0042	0.0055
N1Pa	—0.0007	+0.1145	+0.0782	+0.1245	+0.1415
N1Pb	—0.0042	+0.1165	+0.0772	+0.1262	+0.1450
diff.	0.0035	0.0020	0.0010	0.0017	0.0035
N1Sa	—0.0035	+0.1152	+0.0772	+0.1250	+0.1420
N1Sb	—0.0047	+0.1117	+0.0710	+0.1220	+0.1392
diff.	0.0012	0.0035	0.0062	0.0030	0.0028
N1Ha	—0.0057	+0.0982	+0.0627	+0.1100	+0.1262
N1Hb	—0.0060	+0.1060	+0.0660	+0.1150	+0.1335
diff.	0.0003	0.0078	0.0033	0.0050	0.0073
Maximum diff. —0.0078	Minimum diff. —0.0003	Average diff. —0.0032			
	A Cement—10 Per Cent Substitution				
Aa	—0.0012	+0.0817	+0.0440	+0.0870	+0.1062
Ab	—0.0027	+0.0847	+0.0445	+0.0907	+0.1097
diff.	0.0015	0.0030	0.0005	0.0037	0.0035
A1Fa	—0.0007	+0 0875	+0.0522	+0.0960	+0.1140
A1Fb	—0.0060	+0.0857	+0.0467	+0.0952	+0.1137
diff.	0.0053	0.0018	0.0055	0.0008	0.0003
A1Pa	—0.0020	+0.0917	+0.0557	+0.1005	+0.1185
A1Pb	—0.0010	+0.0905	+0.0532	+0.1012	+0.1185
diff.	0.0010	0.0012	0.0025	0.0007	0.0000
A1Sa	—0.0005	+0.0920	+0.0530	+0.1005	+0.1165
A1Sb	—0.0022	+0.0900	+0.0502	+0.0967	+0.1122
diff.	0.0017	0.0020	0.0028	0.0038	p.0043
A1Ha	+0.0007	+0.0810	+0.0455	+0.0907	+0.1055
A1Hb	—0.0040	+0.0787	+0.0420	+0.0885	+0.1035
diff.	0.0047	0.0023	0.0035	0.0022	0.0020
Maximum diff.—0.0055	Minimum diff.—0	Average diff.—0.0024			

TABLE 3—CONTINUED

Age of Specimen Days	7	142	156	201	339
Specimen	N Cement—30 Per Cent Substitution				
N3Fa	—0.0007	+0.1080	+0.0790	+0.1232	+0.1382
N3Fb	—0.0015	+0.1202	+0.0860	+0.1317	+0.1485
diff.	0.0008	0.0128	0.0070	0.0085	0.0103
N3Pa					
N3Pb	+0.0005	+0.1377	+0.1030	+0.1542	+0.1642
diff.					
N3Sa	—0.0017	0.1392	+0.0962	+0.1460	+0.1637
N3Sb	—0.0062	+0.1360	+0.0920	+0.1452	+0.1592
diff.	0.0045	+0.0032	0.0042	0.0008	0.0045
N3Ha	—0.0035	+0.0920	+0.0584	+0.0967	+0.1165
N3Hb	—0.0042	+0.0945	+0.0602	+0.1015	+0.1220
diff.	0.0007	0.0025	0.0018	0.0048	0.0055

Maximum diff.—0.0128 Minimum diff.—0.0007 Average diff.—0.0053
Grand average difference—0.0033

undoubtedly dried out quickly, this curing was thought to be comparable to that obtained in ordinary construction. If the drying were more rapid, the initial curing was probably more efficient than that of the usual job. For this curing condition, the average strengths of the specimens made with the S and H materials substituted for 30 per cent of the cement by weight were 66, 87 and 97 per cent of the plain mortar strength at 7, 28 and 90 days respectively, whereas the corresponding figures for F and P were 40, 50 and 54.

Since the rate of the reaction between the silica and alumina of the pozzuolanas and the lime of the cement is low, it is to be expected that strength comparisons would be more favorable for the blended cements for long periods of moist curing. This is indicated by the slope of the graphs of Fig. 2, particularly by those for the 30 per cent blends. It is significant that pozzuolanic cements and blends of portland cement and pozzuolanic materials have been used almost exclusively for hydraulic structures

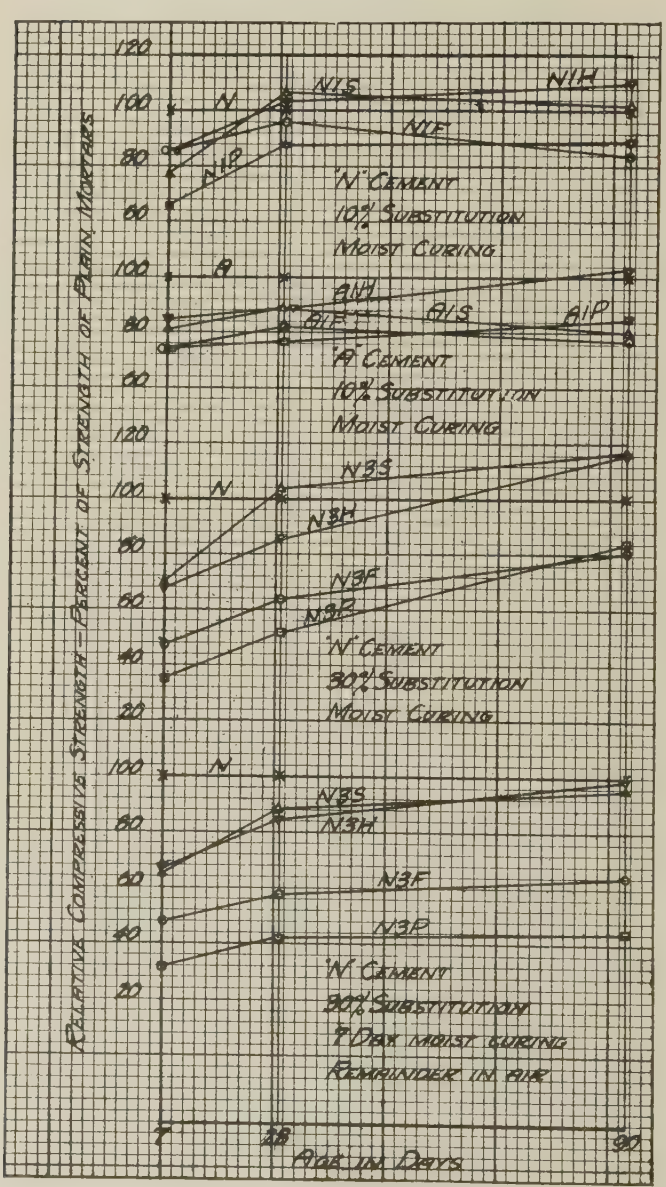


FIG. 2—COMPRESSIVE STRENGTHS OF 1:3 MORTARS MADE WITH BLENDED CEMENTS RELATIVE TO PLAIN MORTARS

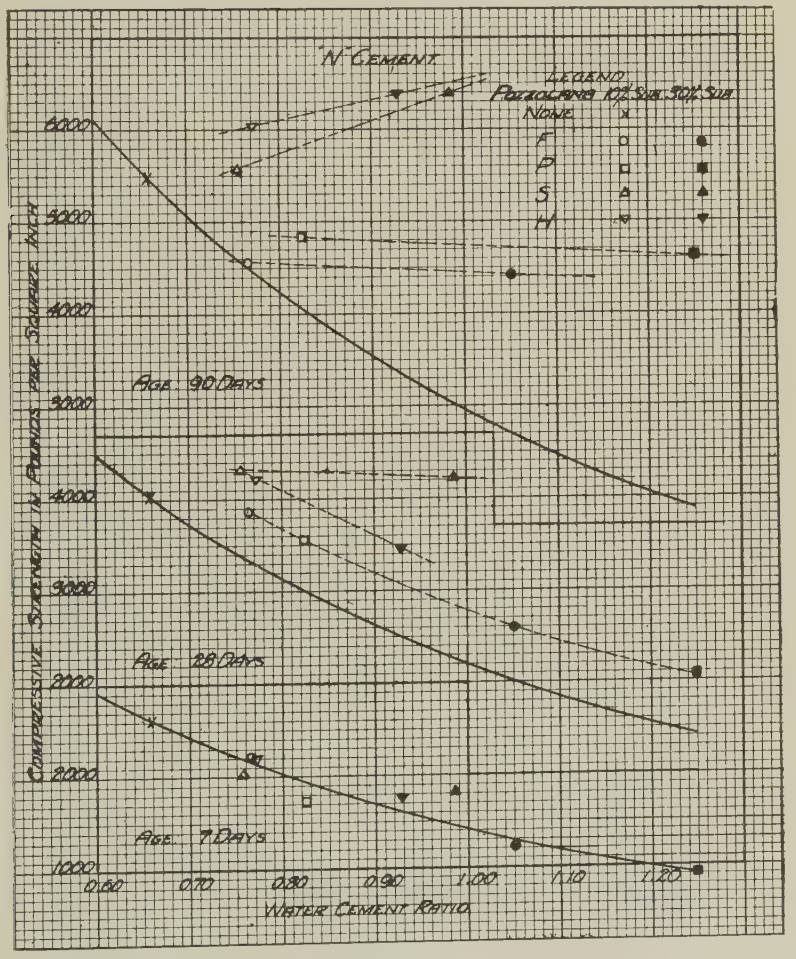


FIG. 3—WATER-CEMENT RATIO STRENGTH RELATIONS FOR BLENDS OF POZZUOLANAS WITH "N" CEMENT

(tunnel linings, aqueducts, sea-walls and the like) in which the conditions are favorable for continued moist curing.[2]

It is at once evident from Fig. 2 that admixtures S and H are more desirable than F and P, particularly for the shorter periods of moist curing and for air curing following a short period of moist curing. When used with N cement, the 30 per cent substitution resulted in greater reductions in strength at early ages, but for continuous moist curing gave strengths at 90 days equal to or greater than those for 10 per cent substitution. The fact that blends of 30 per cent of S and H gave strengths 17 per cent and 16 per cent greater respectively than plain cement is particularly notable.

Water-cement Ratio Relations

In Figs. 3 and 4 the water-cement ratio strength relations are shown for the moist cured specimens. The dotted lines of Fig. 3 joining the points for the 10 and 30 per cent blends have no special significance, but are shown only as an aid in reading the diagrams. The solid lines are believed to approximate the relations for plain mixes. The data for the solid lines were obtained in the following manner: The 7 day strengths were plotted on semi-log paper and a straight line drawn through them. The ordinates for the 28 day and 90 day curves were then obtained by multiplying the 7 day ordinates by the ratios of the 28 day and 90 day to 7 day strengths of the plain specimens. These ratios were 1.55 and 2.09 for N cement and 1.49 and 1.73 for the A cement. Corresponding ratios given by Gilkey[6] for the average portland cement are 1.52 and 1.94. This procedure assumes that the pozzuolanic materials have caused no increase in strength at 7 days over plain specimens made with equal water-cement ratios. If the blended specimens were stronger than corresponding plain specimens (equal water-cement ratios) at 7 days as is likely (see Fig. 5), then the solid lines should slope to the right at a greater angle and the pozzuolanic effect would be greater than that indicated in Figs. 3 and 4.

In general, the divergence from the solid lines increases with increase of age and per cent of substitution. The 10 per cent

[6] "A Method for Predicting Concrete Strengths with Increased Precision." Proc. A. C. I., Vol. XXIV, 1928.

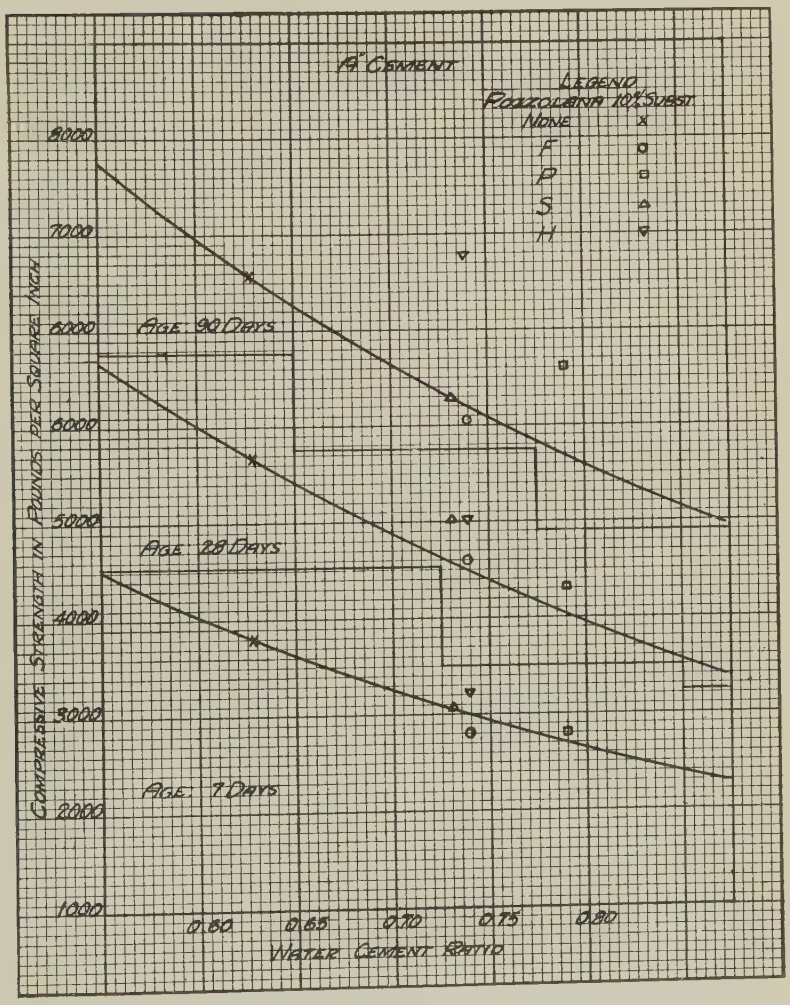


FIG. 4—WATER-CEMENT RATIO STRENGTH RELATIONS FOR BLENDS OF POZZUOLANAS WITH "A" CEMENT

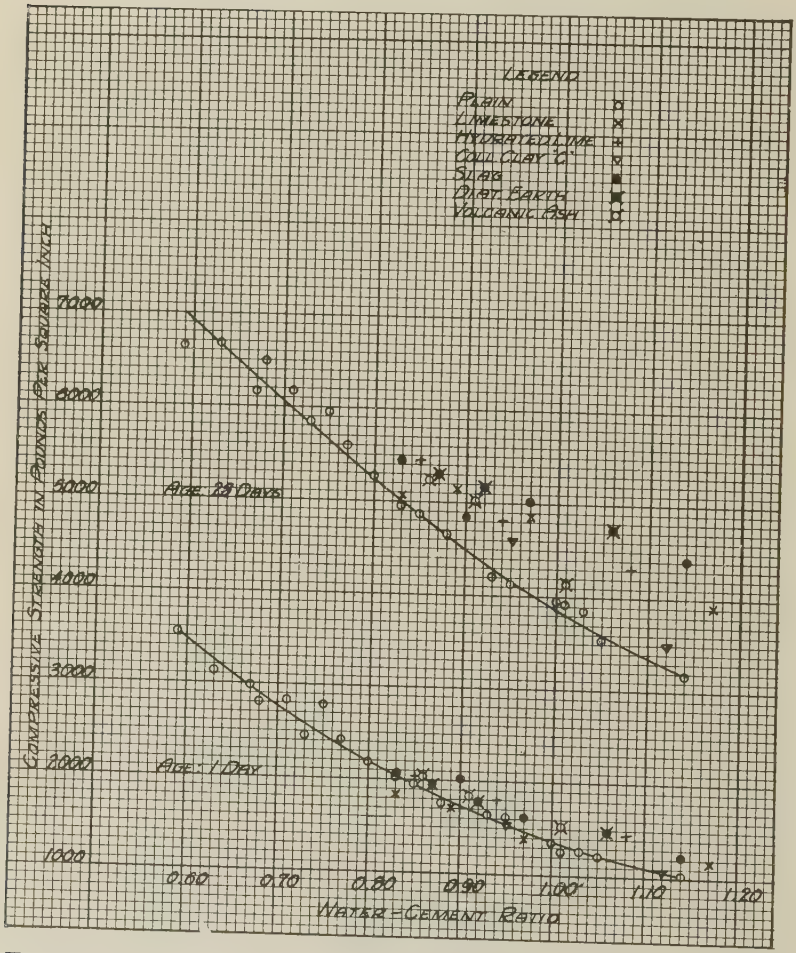


FIG. 5—WATER-CEMENT RATIO STRENGTH RELATIONS FOR CONCRETES CONTAINING VARIOUS ADMIXTURES. DATA FROM TABLES II AND III OF "TESTS OF POWDERED ADMIXTURES IN CONCRETE." BY DUFF A. ABRAMS, PROCEEDINGS A. S. T. M., VOL. 29, 1929

blends of F with N cement and F and S with A cement fall on the solid line at 90 days and yet are above it at 28 days. This does not appear to be in line with the data for the other blends. This may be due to the limited number of specimens tested or may be explained in the following manner:

Since the pozzuolanic materials are chemically active, they will affect the hydration of the cement and we have in effect a new cement having strength-age and water-cement ratio strength relations peculiar to itself just as is the case with different brands of portland cement. Regarding the blends in this way, it would not be unreasonable to expect different percentages of pozzuolana to give strength-age curves differing in shape as well as in position. Since the strength-age curves for different brands of portland cement have been shown to cross,[7] the positions of the points for the 10 per cent blends of F and N cement and F and S with A cement at 90 days might be explained analogously.

Fig. 5 was prepared from data in Tables 2 and 3 of "Tests of Powdered Admixtures in Concrete."[4] The general relations of the data are similar to those of Figs. 3 and 4, the majority of the points falling reasonably close to the curve for the plain specimens at 1 day whereas at 28 days the mixes containing admixtures fall well above the corresponding 28 day curve. These data seem to show that the strengths for equal water-cement ratios are affected by any finely divided material, not just those commonly regarded as pozzuolanic. The high positions of the points for mixes containing limestone certainly cannot be explained by pozzuolanic action.

Figs. 3, 4 and 5 might be misinterpreted to indicate that greatly increased strengths are obtained from the use of admixtures. Fig. 2 shows clearly that such is not the case when the comparison is made on the basis of the same consistency, particularly at early ages and for ordinary periods of curing.

SHRINKAGE AND WEIGHT TESTS

Data from these tests are given in Table 4. The linear change-age relations are shown in Figs. 6, 7 and 8. In the legends shown the numbers 1 and 3 signify 10 per cent and 30 per cent substitution of pozzuolana for cement. The significance of the various

[7] "Basic Principles of Concrete Making." F. R. McMillan. See Fig. 11, page 26.

TABLE 4—SHRINKAGE AND WEIGHT TESTS.

Specimens—2 x 2 x 10 in. beams. Gauge length—8 in. Instrument—Berry Strain Gauge Mix—1:3 by weight.
Pozzuolanas substituted for cement and blended with cement prior to mixing mortars.
Water-cement ratios calculated as in Table 2.
Curing Conditions—1 day molds, 1 day to 7 days in moist room, 7 days to 142 days in air of laboratory, 142 days to 156 days under water, 156 to 339 days in air. Initial Reading taken at 1 day.

Pozzuolanas	Water-Cement Ratio (Vols.)	Ratio—Water to Cement + Pozzuolana by Weight	Relative Linear Change in Millionths			Linear Change Ratios			Per Cent Change in Weight			Weight Change Ratios		
			1—7 Days Moist Room	7—142 Days Air	142—156 Days Water	156—201 Days Air	1—7 Days Moist Room	7—142 Days Air	142—156 Days Water	156—201 Days Air	1—7 Days Increase	7—142 Decrease	142—156 Increase	156—201 Decrease	1—7 Days Increase	7—142 Decrease	142—156 Increase	156—201 Decrease
N Cement—10 Per Cent Substitution by Weight																		
None	0.66	0.437	41	1004	342	509	100	100	100	100	0.31	4.53	4.49	3.76	100	100	100	100
F	0.76	0.457	59	1093	369	496	144	109	108	97	0.54	5.40	4.87	3.69	175	119	105	98
P	0.82	0.492	25	1180	377	476	61	117	110	93	0.36	5.69	5.29	4.09	116	126	118	109
S	0.75	0.452	41	1176	394	494	100	117	115	97	0.38	4.93	4.68	3.59	123	109	104	96
H	0.77	0.462	59	1080	377	481	144	107	110	94	0.54	5.29	4.89	3.86	175	117	109	103
A Cement—10 Per Cent Substitution by Weight																		
None	0.63	0.417	20	852	390	446	100	100	100	100	0.44	4.07	4.13	3.29	100	100	100	100
F	0.74	0.442	34	899	371	461	170	106	95	103	0.09	4.52	4.46	3.44	20	111	108	105
P	0.79	0.472	15	926	366	463	75	109	94	104	0.42	4.81	4.83	3.85	95	118	117	117
S	0.73	0.437	14	924	394	470	70	108	101	105	0.43	4.30	4.25	3.51	98	106	104	107
H	0.74	0.442	17	814	361	458	85	95	93	103	0.55	4.19	4.20	3.56	125	103	102	108
N Cement—30 Per Cent Substitution by Weight																		
None	0.66	0.437	41	1004	342	509	100	100	100	100	0.31	4.53	4.49	3.76	100	100	100	100
F	1.05	0.482	11	1152	316	450	27	115	92	88	0.24	6.15	5.87	4.24	77	136	131	113
P	1.25	0.592	+5	1372	348	513	−12	136	102	101	0.72	7.96	7.05	5.68	232	176	157	151
S	0.98	0.457	40	1460	435	515	98	145	127	101	0.34	4.97	4.79	3.65	110	110	107	97
H	0.93	0.432	39	971	341	399	95	97	100	78	0.41	5.54	5.13	3.86	132	122	114	103

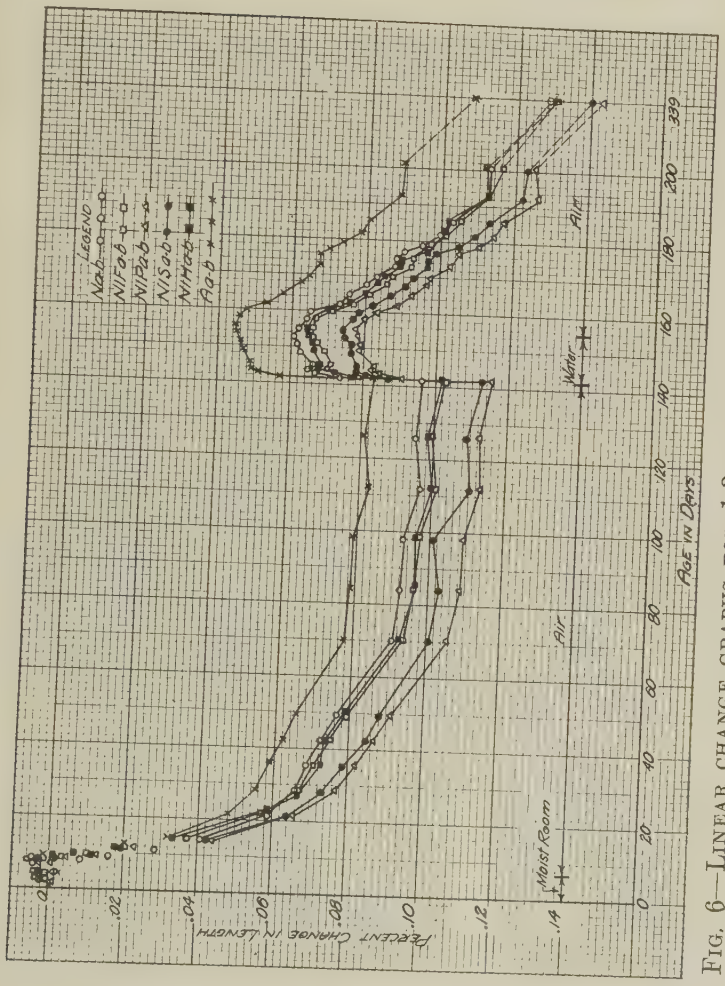


FIG. 6—LINEAR CHANGE GRAPHS FOR 1:3 MORTARS MADE WITH "N" CEMENT AND 10% SUBSTITUTIONS OF POZZUOLANAS.

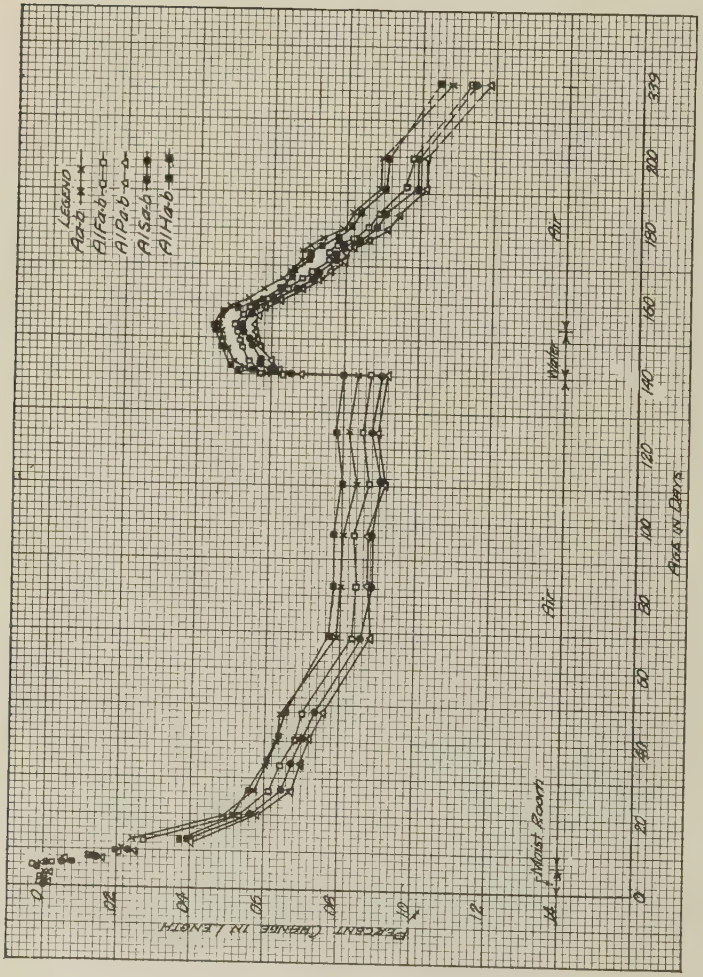


FIG. 7—LINEAR CHANGE GRAPHS FOR 1:3—MORTARS MADE WITH "A" CEMENT AND 10% SUBSTITUTION OF POZZUOLANAS

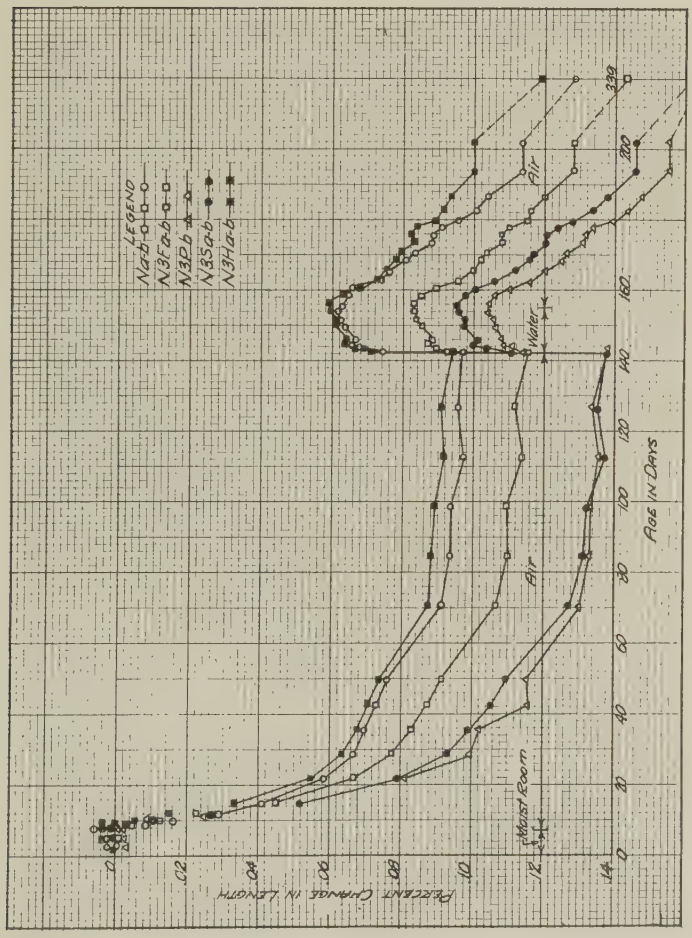


FIG. 8—LINEAR CHANGE GRAPHS FOR 1:3 MORTARS MADE WITH "N" CEMENT AND 30% SUBSTITUTION OF POZZUOLANAS

letters has already been given. The curing conditions for these specimens are given at the head of Table 4 and indicated on the figures. Readings were taken frequently to 201 days. The final reading was taken at 339 days, there being no readings between 201 and 339 days. Weights were taken each time shrinkage measurements were made.

Shrinkage Tests

From Figs. 6, 7 and 8, it will be seen that the shrinkage of the mortars made with blends of H pozzuolana were less for 10 per cent substitution with A cement and 30 per cent substitution with N cement and more for 10 per cent substitution with N cement than that of the corresponding plain mixes. The differences were small, however, and the linear change of these blends may be considered equal to that of the plain mixes. All other blends showed increased shrinkage. The shrinkage curve for the plain mix of A cement is plotted on Fig. 6 to show the effect of the brand of cement. Comparison of Figs. 6 and 7 will show that the difference between the graphs for the two plain mixes is about the same as the maximum difference between the plain and the 10 per cent blended cements. In general, the shrinkage increased with increase in the ratio of effective water to cement plus pozzuolana, the mixes containing S being exceptions to this rule.

The movements of the various mixes relative to those of the corresponding plain mixes are given by the linear change ratios of Table 4. These ratios are less for the shrinkage from 156 to 201 days than from 7 to 142 days particularly for those having 30 per cent substitutions of pozzuolanas. This indicates that with age the volumetric changes may decrease for the blended cements. This is the only evidence given by these tests that shrinkage defects may be less for the blended cements.

Weight Tests

The data from these tests are also given in Table 4. The weight-age graphs (not shown) were similar in shape to the linear change-age graphs. However, the relative positions of the various graphs were different as may be seen from the ratios of Table 4. The order of the weight changes from 7 to 142 days, going from the least to the greatest, is N, N3S, N3H, N3F and

N3P whereas the order of the corresponding volumetric changes is N3H, N, N3F, N3S and N3P.

The increase in weight for the 14 days immersion from 142 to 156 days is the absorption from the air dry condition. It will be seen from Table 4 that the absorptions were greater for the blended cements in all cases.

CONCLUSIONS

While the number of specimens and materials tested is too limited for general conclusions, the following statements appear to be warranted.

For the materials and methods recorded in this report:

1. For equal consistencies, blended cements required more water per unit of cementing material than portland cement (there was one exception, see text).

2. The compressive strengths of mortars made with the blended cements were considerably less at early ages than those of portland cement mortars of the same consistency. At later ages (90 days and greater), with continued moist curing and suitable materials greater strengths may be obtained by use of blended cements. For short curing periods such as are used on ordinary construction, the use of the blended cements resulted in lower compressive strengths.

3. During the period of initial drying, the shrinkage of the blended cement mortars was greater than the shrinkage of the portland cement mortars (two exceptions, see text).

4. The effect of pozzuolanic materials varied with different brands of portland cement.

Readers are referred to the JOURNAL *for April 1931 for discussion which may develop. Such discussion should reach the Secretary by March 1, 1931.*

Tests of Bonding of Floor Finish to Slabs of Haydite and Gravel Concrete

BY F. E. RICHART* AND V. P. JENSEN†

INTRODUCTION

The use of reinforced Haydite concrete for floor slabs has naturally raised the question as to the ability of a finish course of mortar or terrazzo to bond to the slab. Some difficulties have been anticipated by designers due to the tendency for coarse Haydite particles to rise to the surface of the base course and due to the differences in elastic behavior of the topping and base concretes in which the first has a high modulus of elasticity and the second a low one. Questions of this sort have usually been answered through experience with a floor in service and this is without doubt the most satisfactory and convincing answer. The writers know of no laboratory tests that have been made on the bonding of floor finish; such tests could hardly be expected to include differential shrinkages of the two courses over large floor areas, or the effect of hard service, such as heavy trucking, impact, abrasion or vibration. However, in an attempt to devise a test that would show whether there were obvious defects in the bonding, and show them within a short time, the following simple tests were made.

Small reinforced concrete slabs, consisting of a 5-in. base course and ¾-in. finish were tested in flexure on a 4-ft. span with ⅓-point loading. With this arrangement there is produced a relatively high horizontal shear tending to separate base and finish courses in the outer thirds of the span. It was felt that if this type of specimen could be tested to failure without crack-

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ing the topping from the base it would indicate that no inherent defects could be attributed to the bonding of the two courses. For comparative purposes, slabs were made using both gravel and Haydite concretes for the base course.

The tests were made at the Materials Testing Laboratory of the University of Illinois as a part of an investigation being carried on in cooperation with the Western Brick Co., Danville, Illinois, manufacturers of "Western" Haydite aggregates. The work is being carried on under the administrative direction of Dean M. S. Ketchum, Director of the Engineering Experiment Station, and Prof. M. L. Enger, Head of the Department of Theoretical and Applied Mechanics, University of Illinois. The Western Brick Co. is represented by an Advisory Committee consisting of I. N. Doughty, Manager of Research, and Frank Payne, Engineer. All of the Haydite aggregates used in the tests were furnished by the Western Brick Co.

OUTLINE OF TESTS

Twelve slabs were made and tested. The design details and dimensions are given in Fig. 1. Two types of concrete were used in the 5-in. slab base; one made with sand and gravel aggregates, the other with fine and coarse Haydite aggregates. Two mixes of each concrete were used, (1) a fairly rich structural mix, 1:2¼:2¾ by volume, and (2) a lean mix, 1:4½:5½, approximating floor fill. Three slabs and three 6 by 12-in. control cylinders were made of each mix. The ¾-in. finish or topping for all slabs was a 1:1:2 mix made with sand and pea gravel using a water-cement ratio of 0.67.

MATERIALS

Universal portland cement passing standard specifications was used. Average tensile strengths of 1:3 standard Ottawa mortar briquets were 290 and 390 lb. per sq. in. at the ages of 7 and 28 days, respectively.

A well graded sand from Lincoln, Ill., having a fineness modulus of 2.70 was used in the gravel concrete bases and in the finish course. Two sizes of Wabash river gravel from Covington, Ind., were used. A pea gravel, with a fineness modulus of 5.98, was used in the finish course and a mixture of this gravel with a

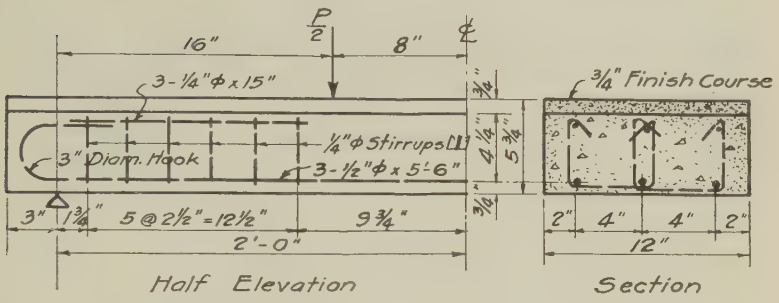


FIG. 1—DETAILS OF TEST SLABS

¾-in. grading, having a fineness modulus of 6.12 was used in the slab concrete.

The Haydite aggregates were the "Western" fine or "A" size, 0—$\frac{3}{16}$-in., with fineness modulus 2.33, and the coarse or "C" size, $\frac{7}{32}$—¾-in., with fineness modulus 5.89.

Each test slab was reinforced with three ½-in. deformed round bars of intermediate grade running the full length of the slab and ending in hooks bent to a 1½-in. radius. These longitudinal bars were tied together in a unit with a double system of ¼-in. round U-stirrups of structural grade spaced 2½-in. apart in the outer thirds of the slab. A typical unit is shown in Fig. 2. None of the reinforcement projected into the finish course of the slab.

MAKING OF SLABS

The slabs were poured in bottomless wooden forms on a sheet of building paper resting on the laboratory floor. Slight leakage



FIG. 2—TYPICAL REINFORCEMENT UNIT



FIG. 3—BASE SLABS

of water and mortar occurred at the bottom edges, particularly with the 1:10 concretes despite the fact that these concretes gave low slumps. The base concrete was mixed in a one-bag batch mixer, hauled to the form in wheel-barrows, shovelled into place and spaded uniformly around the reinforcing steel. The concrete was struck off to a level ¾-in. below the edges of the form. Some settlement occurred after this screeding, especially in the 1:10 mixtures. The settlement was more noticeable in the 1:10 Haydite than in the gravel concretes. After the concrete had stiffened, the surface was struck with a wood float and left exposed to the air until the following day. Three forms are shown in Fig. 3, with the base slabs struck off ready for the finish course.

The finish course was made and placed in general accordance with the recommended practice given in the Report[1] of Committee 802 on Concrete Floor Finish, of the American Concrete Institute. The chief departure from the recommended practice was in the use of a fine sand which was permitted because it was felt that the primary reason for using a coarse sand for floor finish is to improve the wearing quality of the surface, a quality not to be measured in these tests, and that any change in the physical characteristics of the topping, caused by the use of a fine instead of a coarse sand, would have little effect upon the test results.

Twenty-four hours after the base slab was poured, the surface was prepared for the finish course. This was done by wetting

[1] "Good Practice in Concrete Floor Finish" John G. Ahlers, JOURNAL of the Am. Con. Inst., March. 1930, *Proceedings*, Vol. 26, p. 520.

the surface thoroughly and then brushing on a thin coat of neat cement mortar with a wire brush, care being taken to mop up any pool of surface water before brushing in the mortar. The finish was placed immediately after the application of the mortar coat. It was tamped uniformly over the surface of the slab and into the corners of the form to insure a full surface of adhesion in every slab. The surface was then struck off level with the edges of the forms. and compacted and smoothed with a wood float. The surface was trowelled with a steel trowel one to two hours after being struck off.

The average measured thickness of finish used with the four concretes was as follows: 1:5 gravel, 0.74 in.; 1:10 gravel, 0.75 in.; 1:5 Haydite, 0.75 in.; 1:10 Haydite, 0.90 in.

CURING

The forms were stripped one day after the finish course was placed, and the slabs were stored under wet burlap for about five days; they were then removed to the moist room for standard moist curing at 70° F. All cylinder molds were stripped after 48 hours and the cylinders cured in the moist room until the time of test, when they were tested moist.

TESTING OF SLABS

The slabs were removed from the moist room when 28 days old and tested within 6 hours after removal. Loose moisture was wiped from the surfaces and a coating of whitewash was applied to the sides and ends to aid in the detection of cracks. A very marked feature of the 1:10 Haydite concrete slabs was that the moisture present in the base of the slab kept coming to the sides so rapidly that the whitewash was kept from drying during the day of the test.

A general view of the testing apparatus is shown in Fig. 4, which includes a typical slab after test. The work was done in an Olsen testing machine of 300,000-lb. capacity. The load was applied through a spherical bearing block to a grillage made up of four 4-inch channels and thence through rollers and bearing plates to the third-points of the span. The supports permitted free rotation at both ends and horizontal movement at one end. All bearing plates were set in plaster of paris. Load was applied

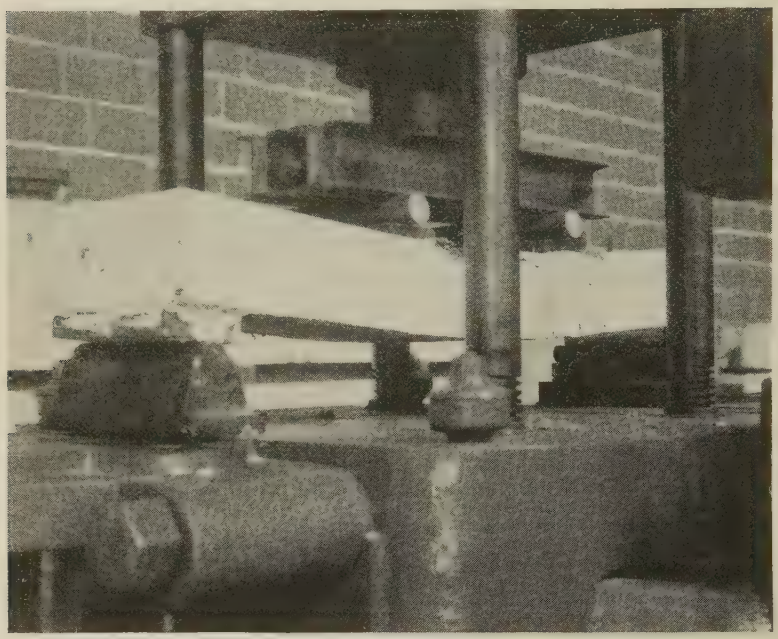


FIG. 4—TESTING APPARATUS SHOWING TYPICAL SLAB AFTER TEST

in increments of 500 to 1000 lb. at the slowest testing speed of the machine. A close watch was kept for cracking of the slab and any indication of loosening of the topping from the base, but no strain measurements were taken. After the steel had passed its yield point, as indicated by a dropping of the weighing beam and opening of tension cracks, a faster test speed was maintained until final rupture occurred.

The modulus of elasticity of the concrete was determined from stress-strain measurements on the control cylinders.

PHENOMENA OF TESTS

With one exception the slabs failed initially by tension in the longitudinal steel. The single exception was a Haydite slab of 1:10 mix which failed initially by diagonal tension at a load of 13,900 lb. In this slab the yield point of the longitudinal steel was reached at a load of 15,700 lb. and the ultimate load carried was 16,300 lb. At failure, a piece split off of the side of the slab

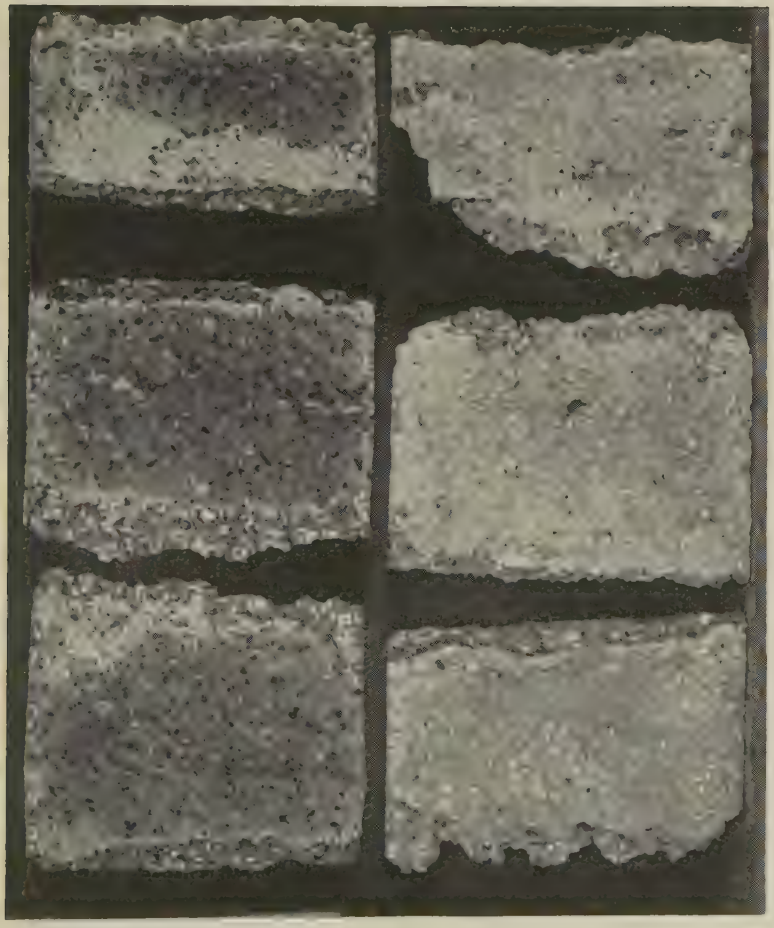


FIG. 5—BOTTOM VIEW OF PIECES OF FINISH WHICH CRACKED OFF AT FINAL FAILURE.

(Top three from slabs of Haydite base; bottom three from slabs of Gravel base.)

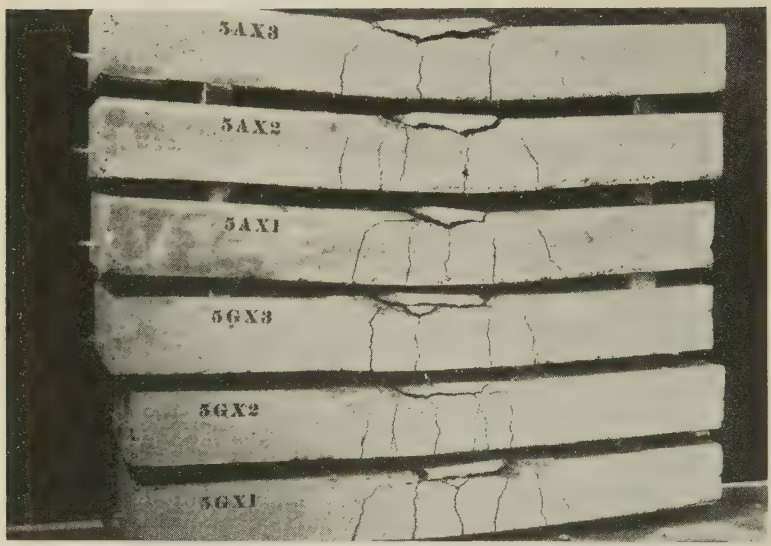


FIG. 6—ALL SLABS OF 1:5 MIX AFTER TEST
(Top three slabs of Haydite base; bottom three slabs of gravel base.)

(No. 10AX3, Fig. 7) near the end, and it was evident that a large amount of slipping of bar and compression of the concrete inside the hook had occurred. There was also evidence of large distortion of the weak and compressible concrete due to high bearing pressure fro n the cast iron plate at the support. While the large amount of movement of the hooked bar indicates that the diagonal cracking may have been due to slip of bar, splitting at the end of the slab occurred well after the yield point of the longitudinal steel had been reached. Complete test data are given for all slabs in Table 1. In no tests did the finish crack loose from the base before the ultimate load was reached. The finish in both Haydite and gravel concrete slabs remained integral with the base with deflections considerably in excess of one inch on a 4-ft. span. At final failure, with deflections of about 1½ inches, crushing of the ¾-in. finish course occurred in the middle third near one of the load points and a section of the topping split off. The sections which split off from the Haydite slabs had portions of the base adhering to the topping, the frac-

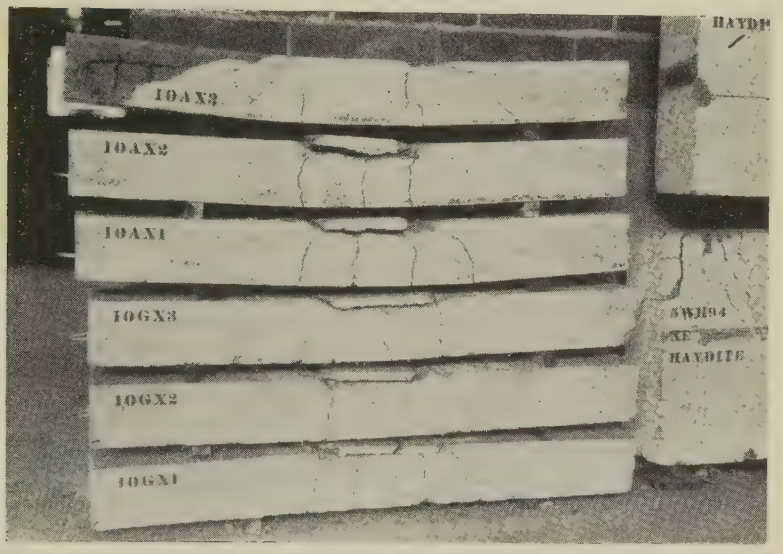


FIG. 7—ALL SLABS OF 1:10 MIX AFTER TEST
(Top three slabs of Haydite base; bottom three slabs of gravel base.)

ture being mostly below the topping. Where the base was of gravel concrete the fracture occurred at the bonding surface between the base and topping. Fig. 5 shows a bottom view of the pieces of topping which split off from the 1:5 concrete slabs. A general view of all the 1:5 slabs is shown in Fig. 6 and of all the 1:10 slabs in Fig. 7. In each figure the upper three slabs are of Haydite concrete and the lower three of gravel concrete.

DISCUSSION OF TEST RESULTS

It is quite evident from these tests that there was no great difference in the bonding of the floor finish to the Haydite and gravel concrete bases. If anything the final crushing failures showed a more definite plane of cleavage between the finish and the gravel concrete than between finish and the Haydite concrete. With care taken in applying the floor finish it is evident that very satisfactory bonding may be secured with either type of base concrete, since in no case was there any separation between finish and base in the outer thirds of the span even at failure of

TABLE I—DATA AND RESULTS OF SLAB TESTS.

Mix by Volume	W/C by Vol.	Slump In.	Flow %	Unit Wt. Fresh Concrete Lb. per Cu. Ft.	Slab Tests			6″ x 12″ Cyl.	
					Slab No.	Load at Yield Lb.	Ulti-mate Load Lb.	Compr. Strength Lb. per Sq. In.	Initial Mod. of Elast. Thousands of Lb. per Sq. In.
Base Concrete Made with Natural Sand and Gravel									
1:2¼:2¾	0.90	5.8	220	146.8	5GX1	15,700	19,700	4390	3720
					5GX2	15,800	18,900		
					5GX3	16,200	18,700		
					Average	15,900	19,100		
1:4½:5½	1.70	1.5	213	142.9	10GX1	15,700	16,700	1260	3220
					10GX2	15 800	17,900		
					10GX3	16,200	18,100		
					Average	15,900	17,600		
Base Concrete Made with Fine and Coarse Haydite									
1:2¼:2¾	1.02	5.3	235	97.1	5AX1	16,000	19,200	3260	1860
					5AX2	16,400	20,300		
					5AX3	15.900	20,100		
					Average	16,100	19,900		
1:4½:5½	2.02	1.9		90.5	10AX1	15,700	18,300	620	920
					10AX2	15,500	17,400		
					10AX3*	*	16,300		
					Average	15,600	17,300		
Finish Course Made with Sand and Pea Gravel									
1: 1 : 2	0.66	5.0		149.2				5540	4030

All slabs except 10AX3 failed initially by tension in the longitudinal steel.
*Slab 10AX3 failed initially by diagonal tension.

the slab. The separation at the point of final crushing of the top surface of the slab might be expected in view of the great distortions and deflections existing in the slab at failure.

An unexpected feature of the tests was found in the fact that the slabs of lean concrete carried practically as much load as those of the rich mixture. It is evident that the strong ¾-in. finish course developed most of the compressive flexural strength of the slabs made with the lean concrete. With roughly 1 per cent of reinforcement in all slabs it appears that the compressive concrete stress was not excessive until the yield point of the reinforcement has been exceeded; it is of interest that a margin of safety existed in the rich layer of the finish course.

It may be concluded that no weakness in bonding between the floor finish and either of the base concretes was indicated by the tests. On the contrary, the tests give support to the practice

of including the floor finish in computing the effective depth of the slab for design purposes. Although the tests indicate that the finish course produced much more slab strength than an equal thickness of base concrete, the writers do not feel that the amount of data secured justify any new method of analysis or design beyond the inclusion of the finish thickness in computing effective depth of slab. So much depended upon the quality of the finish course in the slabs made with lean concrete that any defect in the finish would have greatly decreased the slab strengths. Furthermore, the 1:10 concretes offered such low resistance to local compression that it is surprising that sufficient anchorage was effected to develop the elastic strength of the longitudinal steel.

As noted in the introduction, the tests described give no indication of the effect of shrinkage or of heavy service conditions on a large floor slab; they are quite reassuring in that no obvious defects in bonding of finish to base course was in any way indicated. In the absence of actual service tests of long duration, these experiments indicate that bonding of a floor finish is equally feasible with base slabs of gravel and Haydite concrete, providing careful workmanship is employed.

Readers are referred to the JOURNAL *for April, 1931, for discussion which may develop. Such discussion should reach the Secretary by March 1, 1931.*

Deflection of Reinforced Concrete Members

*Progress Report of Committee 307**

BY T. D. MYLREA,† AUTHOR-CHAIRMAN

PURPOSE

The scope of this report will be limited for the present to a study of the beam and slab type of construction, leaving the flat slab and other forms of construction for future investigation.

An expression for the calculated deflection of a reinforced concrete member may be desirable for one or more of the following purposes: (1) To provide a criterion of acceptability for a given structure, (2) for the computation of the camber required during construction, (3) as a basis for the development of methods of solving statically indeterminate structures. Obviously, to be satisfactory for any of these purposes, an expression should be one from which deflections may be predicted from computed stresses, that is, one which is independent of measured deformations. It is quite possible that such an expression might fulfil one purpose satisfactorily without necessarily being of much use in the others, and for this reason all formulas were studied and data digested with a view to their applicability to any of the above uses. It is hoped that some of the conclusions may be of assistance to the sub-committee of Committee 501 dealing with load tests.

FORMULAS

Two general types of formulas are commonly known—those of Turneaure[1] and those of Maney. Turneaure's formulas are not derived along conventional lines, and hence have not yet

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*This report was on the program of the 26th Annual Convention but in the absence of its author was not presented. It has since been approved by P. J. Freeman and A. H. Saurbrey, as critic members of Committee 307.

[1] "Principles of Reinforced Concrete Construction," by Turneaure and Maurer, 3rd Ed. Chap. VI.

been subjected to a critical study. Maney's formula, as originally propounded,[2] is in the following form:

$$\Delta = C\frac{L^2}{d}(e_s + e_c) \qquad (1)$$

In this formula C is a constant depending upon the method of loading and end restraint, and the rest is a matter of plane geometry. Following the usual procedure in expressing nominal concrete and steel stresses, its author rewrote it[3] as follows:

$$\Delta = C\frac{L^2}{dE_s}(f_s + nf_c) \qquad (2)$$

Since to be of service in predicting deflections a formula based on computed stresses is desirable, Formula 2 is in a much more convenient form than Formula 1.

Prof. Hardy Cross and C. A. P. Turner have proposed a modification of Formula 2, which may be expressed:

$$\Delta = C\frac{L^2 f_s}{d(1 - k)E_s} \qquad (3)$$

However, since k must be computed from the relation between f_s and nf_c there seems to be no special advantage in this form.

Formula 2, therefore, was adopted in making all comparisons.

ASSUMPTIONS AND THEIR EFFECTS

Computed deflections are proportional to the quantity within the parenthesis in Formula 2, and the assumptions used in the flexure formulas will of course affect the value of the terms in this quantity. The value of f_s is based upon the assumption that concrete can take no tension. Yet, at the beginning, only the steel crossing a crack can have its computed value. Later, as bond breaks down, stresses in the steel will be high where computations, indicate the existence of low stress. This condition, by its nature, seems to be closely related to the sum of the crack widths, from which deductions may be made later. End slip of bar will add to crack widths.

The value of n, depending upon a value of E_c which is assumed to be constant from the neutral axis to the extreme compression fiber, will have a much greater effect upon f_c than upon f_s. Consulting any table giving values of j for various values of n will

[2]G. A. Maney, "Relation between deformation and deflection in reinforced concrete beams." Proc. Am. Soc. Test. Materials, 1914, p. 310.
[3]Parcel and Maney, "Statically Indeterminate Structures," p. 48-51, Wiley.

show than for a change of 25 per cent in the values of n the corresponding change in j, and consequently in f_s, is generally less than 2 per cent. And this condition does not change much even when E_c is assumed to be variable, as in the so-called "parabolic formulas. Changes in f_c are inversely proportional to changes in n.

Another factor which materially affects the computed deflection of actual structures, as distinguished from isolated specimens, is the assumed width of T. This has a direct bearing on the magnitude of f_c; and until correct T values can be assigned it is futile to conduct load deflection tests with the object of passing upon the merits of a completed piece of construction. This matter will be given further consideration when available test data have been discussed.

Effect of n upon Δ.

Since Δ varies directly as $(f_s + nf_c)$ it is easy to study the part that n plays in computed deflections. As n increases f_c decreases, and vice versa, so that the variation in the term nf_c is much less than the variation in n. For example, assume a beam in which $f_s = 18{,}000$, $f_c = 800$ and $n = 15$. In this beam $p = 0.0089$ and

$$f_s + nf_c = 18{,}000 + 15 \times 800 = 30{,}000.$$

Now had the value of n been 12, in a beam with this steel ratio, f_s would have been 17,800 and f_c would have been 870.

Here $f_s + nf_c = 17{,}800 + 12 \times 870 = 27{,}840.$

In other words, with a change of 25 per cent in the value of n there is a change in deflection of only 7.75 per cent.

The discrepancy is more noticeable in the case which is more usual in building construction, i. e., where the concrete stresses are low because of wide T-flanges. Consider, for example, a beam in which $f_s = 20{,}000$, $f_c = 500$, and $n = 15$. In this beam $p = 0.0034$ and

$$f_s + nf_c = 20{,}000 + 15 \times 500 = 27{,}500.$$

If the value of n had been 12, then f_s would have been 19,850 and f_c would have been 545. Here

$$f_s + nf_c = 19{,}850 + 12 \times 545 = 26{,}400,$$

and in this case a difference of 25 per cent in the value of n would have occasioned only 4.17 per cent difference in deflection.

In flat slab construction where p is always low, the difference in deflection might be even less.

Now in buildings under test the deflections are of the order of hundredths of an inch. An investigator would be hardy indeed to condemn a building as having concrete 25 per cent below par when his deflections varied only a few per cent from some predetermined few hundredths of an inch. The author advances this point for the consideration of the sub-committee on load tests.

On the other hand, the very fact that deflections are not proportionately influenced by variations in concrete strength is of much value in computing camber and in sustaining confidence in computations involving continuity. Accidental variations in strength will not necessarily vitiate computations.

GENERAL OBSERVATIONS ON AVAILABLE DATA

So far the following literature has been examined: University of Illinois Engineering Experiment Station Bulletins No. 4, 12, 28, 29, 64 and 71; United States Bureau of Standards Technological Paper No. 314, and Deutscher Ausschuss für Eisenbeton Heft No. 38. In addition, the author has at hand the data from the tests of 10 large T-beams. These papers with the exception of Bulletin 314, all deal with isolated simply supported rectangular or T-beams. In Bulletin 71 and Heft 38, end slip of the main reinforcing is recorded, the significance of such slip having by this time become evident. In Bulletin 314 the effects of slip are noted carefully. In some of the beams tested by the author slip was permitted and in others it was prevented. A comparison of deflections is being made.

A general characteristic of nearly all tests is that loads were applied progressively to failure, stopping at various increments just long enough to make the required observations and measurements. University of Illinois Bulletin No. 4 has some valuable curves showing the effect of repeated applications of the same load as well as curves showing what took place when each load following a release was higher than the one previous.

COMPARISON OF ACTUAL AND COMPUTED DEFLECTIONS

Published load deflection curves made from actual test data where loads were carried progressively to failure are all curved. For quick reference, the reader is referred to a typical curve of

this nature on page 281 of the January issue of the JOURNAL of the American Concrete Institute. It will be noted that at low loads, say below 2500 lbs. the curvature is sharp, while the remainder of the curve might almost be represented by a straight line. About the middle of the curve is a point labelled "first crack." This test was made before it was discovered at the University of Wisconsin that actual cracks occurred long before they were visible to the naked eye. On the basis of the reported tensile strength of the concrete used, the first crack probably occurred at a load of between 1000 and 1500 lbs.—i. e. approximately at the beginning of the sharp curvature; and it would appear that the shape of the curve below the 3000 lb. load is largely due to the formation and development of fine cracks. The beam is reported to have failed in tension. Had deflection readings been continued beyond the loads shown, a comparison of similar curves indicates that this curve would have deviated again from the nearly straight line condition bending over to the horizontal and perhaps below it. Such a deviation also accompanies end slip of bar as may be seen in the graphs on pages 229-238 of Bulletin No. 71. It is of interest to note in the tests reported in Bulletin No. 71 that while slip of bar along the middle portions of the beams occurred at very low loads, end slip did not make its appearance until steel stresses approximately as high as those in use for general design purposes had been attained. There is a portion of the curve, then, which may be of special significance.

Turning again to Formula (2), since f_s and nf_c, the computed fibre stresses, are directly proportional to the applied load, the curve of computed deflections should be a sloping straight line passing through the origin. Computations in many of the typical cases indicate that this line intersects the curve of actual deflections at such a point as would give a computed steel stress of from 18,000 to 28,000 lb. per sq. in.—generally at about the point marking first end slip. Where end slip is prevented the higher values obtain.

It would be easy to write an equation representing such a curve as the one shown on page 281 of the JOURNAL of the A. C. I., January, 1930, but an equation of this sort would serve no useful purpose, since no portion of this curve would ever be duplicated in this beam. Suppose, for example, that in testing for the first

time the load had been progressively applied up to 4,000 lb. The load-deflection diagram would be as shown in the figure up to point "A." Upon the release of the load the beam would recover, and if the release were gradual an "unload" deflection diagram would be a straight line from point "A" back to the point of zero load but not to the point of zero deflection. There would be a permanent deflection of about .01 in. as indicated by the "set curve." Experiments by the author, in which loads were repeated as many as 55 times, indicate that if this 4000 lb. load were repeatedly applied and released, the load and unload curves would follow up and down this sloping line. Similarly, for any other load, for instance, the 7000 lb. load, the load and "unload" curve would follow up and down an approximately straight line from about the point marked "first crack" to a point on the deflection axis of about .025 in. The process apparently would be repeated until the magnitude of the load was such that end slip of the bar or progressive permanent deformation of the concrete took place. This condition would be apparent by the fact that repetition of the load would produce a "saw-tooth" curve with the peaks all at the same altitude. If any load were applied once and then released, the "unload" curve would be approximately a straight line, but upon the next application of a higher load, the upper end of the ordinate representing the resulting deflection would lie on the curve given on page 281. It is as yet an open question just to what extent tha permanent deformation of the concrete affects the deflection, and this, of course, leads at once to the study of long-continued loads and plasticity. It is apparent, too, that if a beam has once been subjected to a high load, the load-deflection curve for any subsequent progressively applied load will lie below the typical load-deflection curve and will probably more nearly coincide with the curve of computed deflections regardless of the assumed value of n. If it is possible that a rational equation can be found representing the points on a deflection curve plotted for loads following the application of a high load, it may also be possible, (since the effect of the value of n is not of major importance, and since the value of E_c for concrete stresses usually encountered in design does not vary much from the neutral axis to the extreme compression fibre of a beam), that by the test of a bay in a building

the width of floor slab performing flange duty for a beam may be found deductively from Formula (2). For example, if such a test could now be applied to the George Mason Hotel (see Prof. Slater's article in the January 1930 issue of the JOURNAL of the A. C. I.) with loads below 238 lb. per sq. ft., this building, because of the extreme flange width obtainable, might well serve such a purpose.

BEAMS OF UNUSUAL SHAPE

The above observations are all based upon the tests of rectangular or T-beams. The results reported in Technological paper No. 314 however, seem to indicate that any conclusions drawn above may not apply to beams differing much in shape from the usual rectangular or T-shaped beam. In the beams tested for the Shipping Board, from the very beginning in most cases, the actual deflections exceeded the computed deflections—just the reverse of what has been previously discussed. This held true even when actual measured deformations were substituted in Formula (1), and hence when the agreement ordinarily would have been close throughout the whole range of the test. The authors ascribe this discrepancy between the actual and computed deflections to shearing deflection and vertical opening between cracks. From this it seems possible that the neglect of shearing distortions may have a relatively greater effect in deep concrete beams than in short deep beams of other structural materials, and that such deflections may not mean an unsafe structure. It means, too, that any deductions based on tests of such beams in a completed structure must be made with extreme caution.

CONCLUSION

In presenting this progress report the author has attempted to cover the work already done, and to outline the plan for future work, and will appreciate constructive criticism or suggestions from any interested members.

Readers are referred to the JOURNAL *for April 1931 for discussion which may develop. Such discussion should reach the Secretary, by March 1, 1931.*

Continuous Beams and Frames in Building Construction

BY T. U. BERG*

The object of this paper is to present a ready means of determining the bending moments and shearing forces for continuous beams and frames of approximately equal spans. Except as indicated in Appendixes 1 and 2, this is not a general method of solving rigid frames, but a presentation of simple formulas and constants from which the designer of reinforced concrete can directly read maximum bending moments with due consideration for the influence of dead as well as live loads.

The negative moments at supports for critical loadings can easily be determined from the accompanying table or graphs. (The graphs have all been prepared from values given in Table 3). The problem is then reduced to a statistically determinate one, and the moment curves are drawn in the usual way by combining the negative support moments with the moments due to the loading acting on the simply supported span.

The moment at any support and for any system of loading can be written: $M = C_D \cdot K_D + C_S \cdot K_S$ where K_D and K_S are constants respectively for dead and live loads as given in Table 1, Fig. 2. It will be noted that these constants are similar to what are generally called "moment area constants." They are: 1½ times the area of the moment curve for a simple supported beam divided by the span. For combinations of loadings, determine one value for each type of loading and add them together.

C_D is a coefficient to be read directly from the table (or graph). For convenience, D will be used in the following instead of C_D.

*Structural Engineer, New York City.

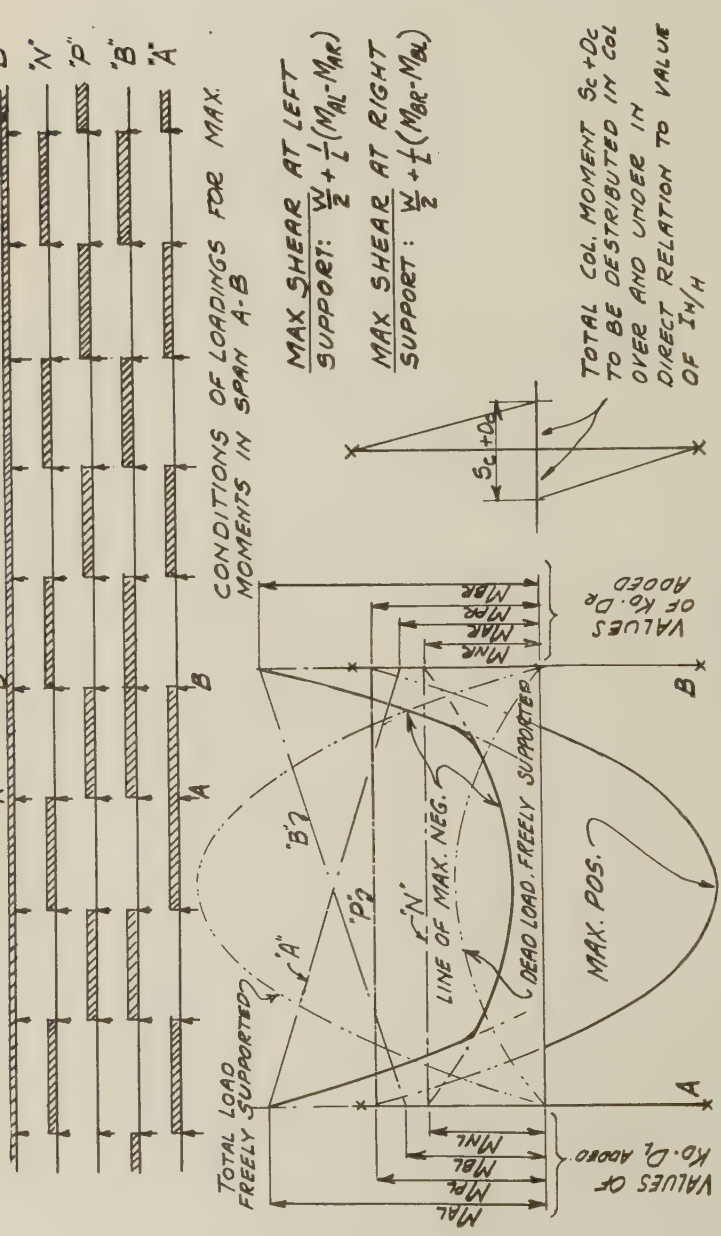

"D"
"N"
"P"
"B"
"A"
A
B
CONDITIONS OF LOADINGS FOR MAX. MOMENTS IN SPAN A-B
MAX SHEAR AT LEFT SUPPORT: $\frac{W}{2}+\frac{1}{L}(M_{AL}-M_{AR})$
MAX SHEAR AT RIGHT SUPPORT: $\frac{W}{2}+\frac{1}{L}(M_{BR}-M_{BL})$
TOTAL COL. MOMENT S_C+D_C TO BE DESTRIBUTED IN COL OVER AND UNDER IN DIRECT RELATION TO VALUE OF I_H/L
S_C+D_C
VALUES OF Ko·DL ADDED
VALUES OF Ko·DL ADDED
TOTAL LOAD FREELY SUPPORTED
"A"
"B'2"
"P'2"
"N"
LINE OF MAX. NEG.
DEAD LOAD FREELY SUPPORTED
MAX. POS.
MAL
MNL
MPL
MBL
MNR
MPR
MBR
MNR
MAR

FIG. 1

TABLE 1

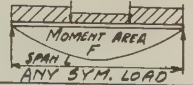
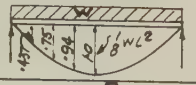
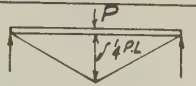
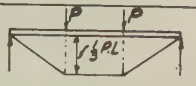
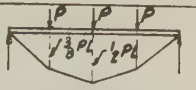
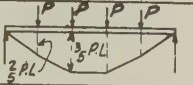
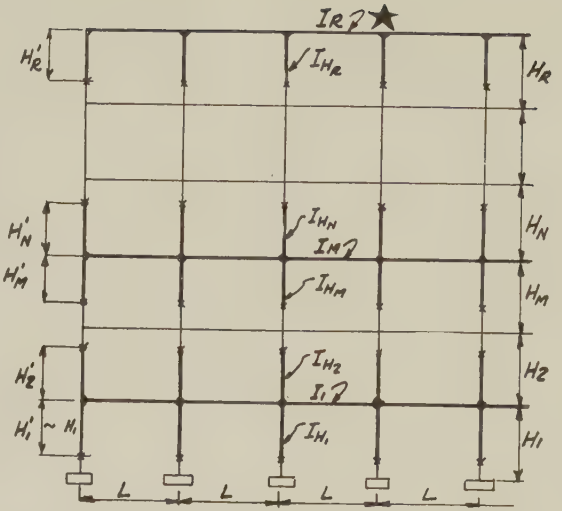
VALUES OF K_0 & K_3	
LOADING	K
	$1.5\frac{F}{L}$
	$\frac{1}{8}WL^2$
	$\frac{3}{16}PL$
	$\frac{1}{3}PL$
	$\frac{15}{32}PL$
	$\frac{3}{5}PL$
	$\frac{5}{32}WL$

TABLE 2

$\frac{I_H}{H'}\frac{L}{I}$	HORIZ. SCALE	$\frac{H'}{H}$
0	0	
.1	1.30	0.76
.2	2.40	"
.3	3.20	"
.4	3.90	0.77
.5	4.50	"
.6	5.10	0.78
.7	5.50	"
.8	5.90	"
.9	6.30	0.79
1.0	6.55	"
1.5	7.70	0.80
2.0	8.50	0.81
3.0	9.50	0.82
4.0	10.00	0.83
6.0	10.60	0.84
8.0	11.00	"
10.0	11.20	0.85
∞	12.20	



FOR DIFFERENT VALUES OF $\frac{I_N}{N}$ FOR COLUMNS BELOW AND ABOVE FLOOR, USE AVERAGE VALUE, $\frac{1}{2}\left(\frac{I_{HM}}{H_M'} + \frac{I_{HN}}{H_N'}\right)$

★ FOR COLS. BELOW FLOOR ONLY USE HALF VALUE ONLY. $I_N/H' = 0.5 \cdot I_{HR}/H_R'$

FIG. 2

C_S is a coefficient depending on the arrangement of live loads, i. e. which spans are loaded or unloaded.

As our aim is to determine maximum values for moments, we must consider the following cases:

Fixed Loads. C_S for live loads of similar nature to dead loads, such as permanent brick walls etc., should be taken the same as for dead loads. $C_S = C_D = D$.

Maximum Positive Moments. This condition occurs for the system of loading as shown in Fig. 1, mark P, i. e. spans alternately loaded and unloaded. For this case we will substitute the mark P for live load coefficient instead of C_S, and the support moment can be written: $M_P = D \cdot K_D + P \cdot K_S$.

Maximum Negative in Middle of Span. See loading as shown in Fig. 1, mark N. Note that the span investigated is unloaded (dead or fixed loads only). All other spans are alternately loaded and unloaded. Use N as mark for coefficient instead of C_S, and we have for support moments: $M_N = D \cdot K_D + N \cdot K_S$.

Maximum Negative Near Support. For a span A-B, the maximum moment at support B will occur for loading same as shown in Fig. 1, mark B, i. e., spans at both sides of support B loaded, other spans alternately unloaded and loaded. Substitute B for coefficient C_S and we have: $M_B = D \cdot K_D + B \cdot K_S$.
In a similar manner we have for support A, loading A, Fig. 1: $M_A = D \cdot K_D + A \cdot K_S$.

In order to draw an accurate curve for moments near A, and B, it is necessary also to determine the moment at the opposite end. Thus, for maximum at B, a moment should also be figured for the other end A. To make this clear, the constants have been given the following marks:

Loading A (maximum moment at support A):
C_S (at left support A) $= A_L$, $M_{AL} = D \cdot K_D + A_L \cdot K_S$
C_S (at right support) $= A_R$, $M_{AR} = D \cdot K_D + A_R \cdot K_S$
Loading B (maximum moment at support B):
C_S (at left support A) $= B_L$, $M_{BL} = D \cdot K_D + B_L \cdot K_S$
C_S (at right support) $= B_R$, $M_{BR} = D \cdot K_D + B_R \cdot K_S$
It should be noted that for each support, values must be determined for M_P, M_N, M_A and M_B, but they are all simple slide rule operations.

K=L/I	LOADING	MARK	2-SPANS		3-SPANS				4-SPANS				5-SPANS						TYPICAL		
			I		I		II		I		II		I		II		III				
			L	R	L	R	L	R	L	R	L	R	L	R	L	R	L	R	L	R	
0	DEAD LOADS	D	0	1	0	.800	.800	.800	0	.857	.857	.571	0	.842	.842	.632	.632	.632	.667	.667	K_D
	ALT. SPANS LOADED	P.N	0	.500	0	.400	.400	.400	0	.429	.429	.286	0	.421	.421	.316	.316	.316	.333	.333	
	MAX. AT SUPPORTS LEFT	A	—	—	—	—	.933	.278	—	—	.964	.143	—	—	.957	.172	.890	.163	.909	.181	K_S
	MAX. AT SUPPORTS RIGHT	B	0	1	0	.933	.278	.933	0	.964	.286	.857	0	.957	.278	.890	.163	.890	.181	.909	
0.10	DEAD LOADS BEAMS	D	.087	.957	.100	.798	.768	.768	.097	.839	.817	.592	.098	.829	.804	.634	.641	.641	.667	.667	K_D
	DEAD LOADS COLUMNS	Dc	.087	0	.100	-0.030	—		.097	-0.022			.098	-0.025		0.007	—		0		
	POSITIVE DISTR.	P	.123	.520	.130	.434	.434	.434	.129	.453	.460	.339	.129	.447	.455	.358	.366	.366	.377	.377	
	NEGATIVE DISTR.	N	+.036	.436	+.030	.364	.334	.334	+.031	.386	.356	.253	+.032	.382	.349	.276	.275	.275	.290	.290	
	MAX. AT SUPPORTS LEFT	A	.123	.520	.130	.434	.896	.306	.129	.453	.923	.205	.129	.447	.918	.224	.867	.222	.884	.220	K_S
	MAX. AT SUPPORTS RIGHT	B	.087	.957	.091	.905	.306	.896	.089	.929	.322	.839	.090	.924	.315	.862	.222	.867	.230	.884	
	LIVE LOADS ON COLS	Sc	.123	.084	.130	-0.102	—		.129	-0.096	.086		.129	-0.098		0.090	—		0.087		
0.40	DEAD LOADS BEAMS	D	.249	.877	.270	.775	.721	.721	.266	.804	.753	.623	.271	.800	.747	.643	.656	.656	.667	.667	K_D
	DEAD LOADS COLUMNS	Dc	.249	0	.270	-0.054	—		.266	-0.051			.271	-0.053		0.013			0		
	POSITIVE DISTR.	P	.321	.554	.334	.495	.495	.495	.333	.502	.516	.435	.338	.500	.514	.442	.455	.455	.458	.458	
	NEGATIVE DISTR.	N	+.072	.323	+.064	.290	.225	.225	+.067	.302	.237	.189	+.067	.300	.233	.201	.201	.201	.208	.208	
	MAX. AT SUPPORTS LEFT	A	.321	.554	.334	.495	.832	.385	.333	.502	.853	.323	.337	.500	.850	.330	.824	.333	.833	.333	K_S
	MAX. AT SUPPORTS RIGHT	B	.249	.877	.256	.850	.385	.832	.253	.862	.395	.803	.253	.860	.391	.825	.333	.824	.333	.833	
	LIVE LOADS ON COLS	Sc	.321	.230	.334	-0.270	—		.333	-0.265	.246		.338	-0.267		0.241			0.250		
1	DEAD LOADS BEAMS	D	.400	.800	.414	.759	.690	.690	.412	.765	.706	.647	.412	.764	.703	.653	.663	.663	.667	.667	K_D
	DEAD LOADS COLUMNS	Dc	.400	0	.414	-0.069	—		.412	-0.059			.412	-0.061		0.010	—		0		
	POSITIVE DISTR.	P	.473	.582	.483	.552	.552	.552	.482	.553	.563	.521	.482	.553	.563	.523	.533	.533	.533	.533	
	NEGATIVE DISTR.	N	+.073	.218	+.069	.207	.138	.138	+.070	.211	.142	.126	+.067	.211	.141	.131	.131	.131	.133	.133	
	MAX. AT SUPPORTS LEFT	A	.473	.582	.483	.552	.775	.467	.482	.553	.786	.436	.482	.553	.785	.437	.776	.439	.781	.439	K_S
	MAX. AT SUPPORTS RIGHT	B	.400	.800	.404	.790	.467	.775	.402	.795	.471	.765	.402	.795	.469	.769	.439	.776	.439	.781	
	LIVE LOADS ON COLS	Sc	.473	.363	.483	-0.413			.482	-0.411	.395		.482	-0.412		0.402			0.400		
10	DEAD LOADS BEAMS	D	.625	.687	.626	.686	.667	.667	.626	.686	.667	.666	.626	.686	.667	.666	.667	.667	.667	.667	K_D
	DEAD LOADS COLUMNS	Dc	.625	0	.626	-0.019			.626	-0.019	—		.626	-0.019		0			0		
	POSITIVE DISTR.	P	.644	.647	.645	.646	.646	.646	.645	.646	.647	.645	.644	.646	.646	.645	.646	.646	.646	.646	
	NEGATIVE DISTR.	N	+.019	.040	+.019	.040	.020	.020	+.019	.040	.020	.020	+.019	.040	.021	.021	.021	.021	.021	.021	
	MAX. AT SUPPORTS LEFT	A	.644	.647	.645	.646	.686	.627	.645	.646	.687	.626	.645	.646	.687	.626	.687	.626	.687	.626	K_S
	MAX. AT SUPPORTS RIGHT	B	.625	.687	.625	.687	.627	.686	.625	.687	.627	.686	.625	.687	.627	.686	.626	.687	.626	.687	
	LIVE LOADS ON COLS	Sc	.644	.608	.645	-0.625			.645	-0.625	.624		.645	-0.625		0.625			0.625		

TABLE 3

K_D and K_S are assumed to be of constant value for all spans, and $D \cdot K_D$ is figured only once for each support. See combined moment curves, Fig. 1. It is convenient to draw one diagram for total load on a simply supported beam to be used for P, A, and B lines, and one diagram for dead loads on a simple supported beam to be used for N lines. With the aid of dividers all moment curves can be drawn in a short time.

Shearing Forces. The maximum shear at any support equals the shear for a simply supported beam plus a value depending on the slope of the A and B *lines.* This addition for left support is $1/L \cdot (M_{AL} - M_{AR})$, and for right support, $1/L \cdot (M_{BR} - M_{BL})$.

Column Moments. Maximum moments for columns are given in Table 3 or in the graphs in the following manner:

Coefficient for dead load, D_C
Coefficient for live load, S_C
Column moment, $M_C = D_C \cdot K_D + S_C \cdot K_S$

Moments have been considered positive if they turn in a clockwise direction.

The Basic Frame. The coefficients D, P, N, A and B are dependent upon the form of the frame and stiffness of beams and columns. The main table No. 3 has been figured for frames shown in Fig. 3 for 2, 3, 4 and 5 spans and for a typical interior bay.

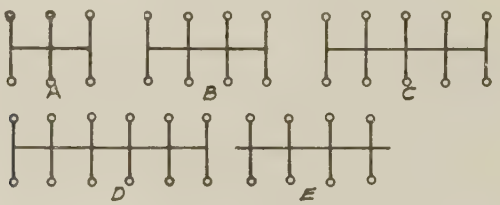


FIG. 3—DIAGRAMS OF FRAMES INCLUDING TWO, THREE, FOUR AND FIVE SPANS, AND TYPICAL INTERIOR BAY

Note the hinged ends of columns for basic frame and see later for use on continuous columns. Each frame has been analyzed for a stiffness ratio of column stiffness I_h/H' to beam stiffness I/L of 0.0 (continuous beams) 0.1, 0.4, 1.0 and 10.0. The resulting

values are given in Table 3 and are also shown in graph form on Figs. 4 to 7 inclusive. The values for $\frac{I_h \cdot L}{H' \cdot I}$ have been drawn on the horizontal axis to a convenient scale. This scale is shown in Table 2, Fig. 2 and its purpose is to make the curves as nearly straight as possible.

Note instructions on Fig. 2 for the use of column stiffness. The stiffness of the exterior columns is frequently different from that of the interior columns. It is recommended that moments for such cases should be adjusted for end bays, using the revised value for column stiffness.

APPLICATION OF THE BASIC FRAME TO CONTINUOUS COLUMNS

The condition as assumed for the basic frame, of column height H' and columns hinged at one end is very rare in building construction. It is necessary to find a relationship between the basic frame and the multiple story building, with story height H.

On Fig. 8A the slope of the elastic curve for a moment $M = 1$ and height H' (the condition similar to the basic frame), is $\phi_1 = \frac{H'}{3EI}$ and on Fig. 8B, the slope for a continuous column (story height H) and for $M = 1$, is

$$\phi_2 = \frac{H}{6EI}\left[3 - \frac{H}{H - B}\right], \text{ for } \phi_1 = \phi_2, \text{ we have:}$$

$$H' = \frac{H}{2}\left[3 - \frac{H}{H - B}\right]$$

The value of B depends on the number of spans, stiffness of columns, etc., but the writer believes that the values given for H'/H in Table 2 are sufficiently accurate for use in practical design. Note that as the story height H is the known value, it will be necessary to make a few trials so that the values for $\frac{I_h \cdot L}{H' \cdot I}$ and H'/H in Table 2 correspond.

The Influence of B Values. Fig. 9 has been prepared to better demonstrate the meaning of B values or H'/H. Although the writer believes that satisfactory results can be reached by using the values H'/H from Table 2 for all types of loadings (for all

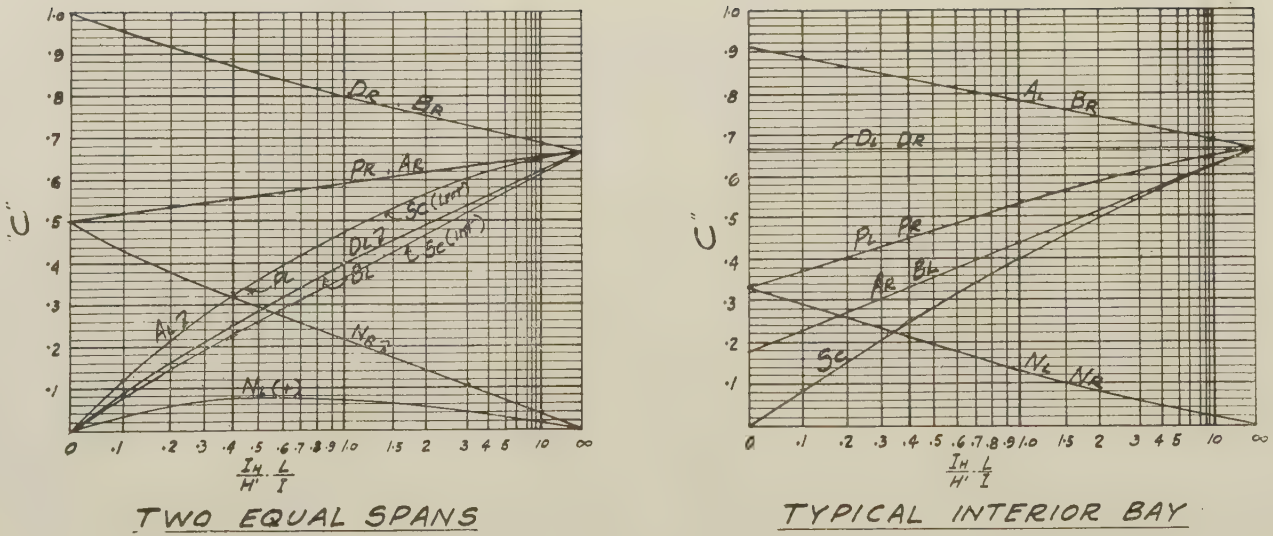


FIG. 4—GRAPH FOR TWO EQUAL SPANS AND TYPICAL INTERIOR BAY

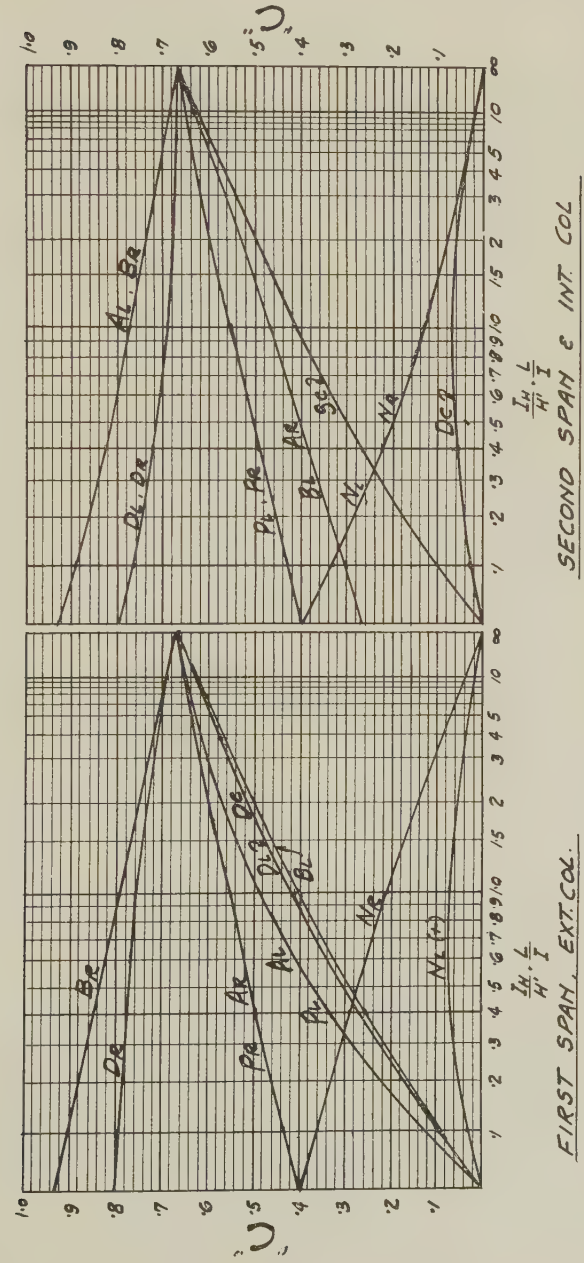


FIG. 5—GRAPH FOR THREE EQUAL SPANS

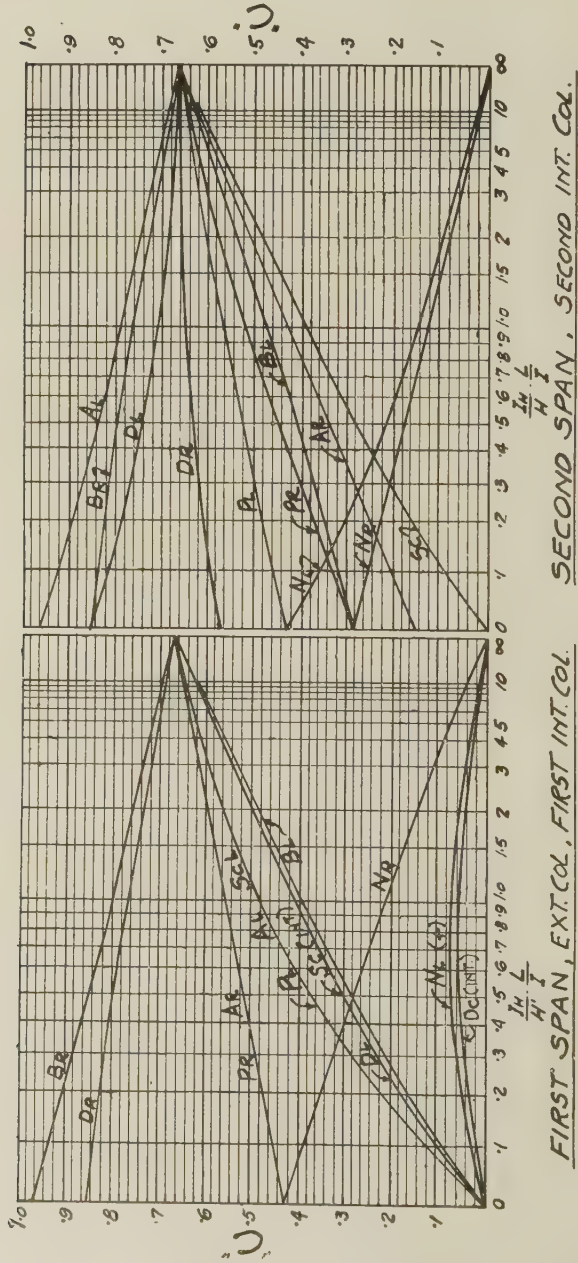


FIG. 6—GRAPH FOR FOUR EQUAL SPANS

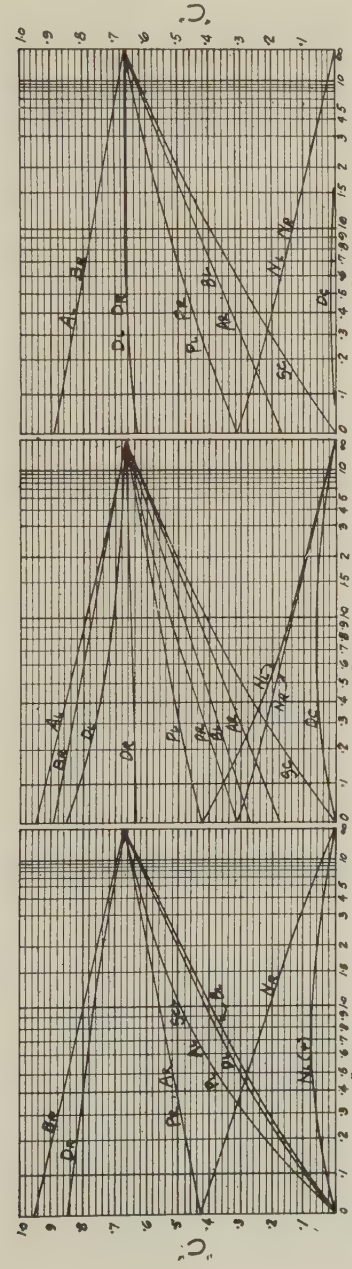


FIG. 7—GRAPH FOR FIVE EQUAL SPANS

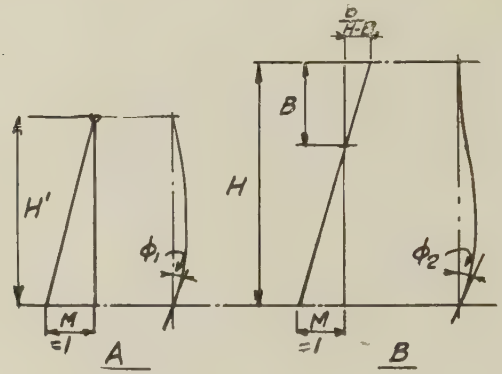


FIG. 8—A AND B

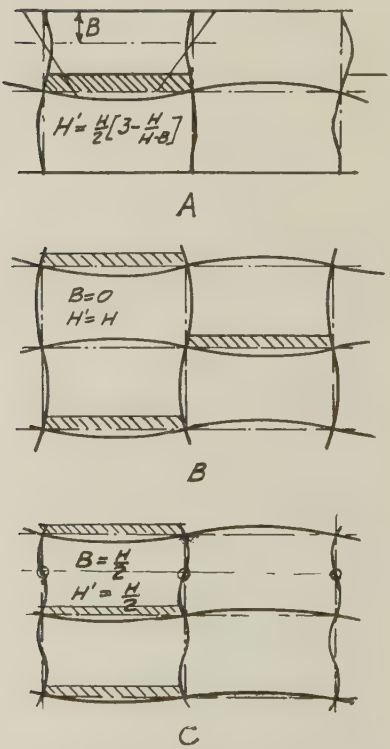


FIG. 9—POINTS OF CONTRAFLEXURE FOR DIFFERENT ARRANGEMENTS OF LOADING

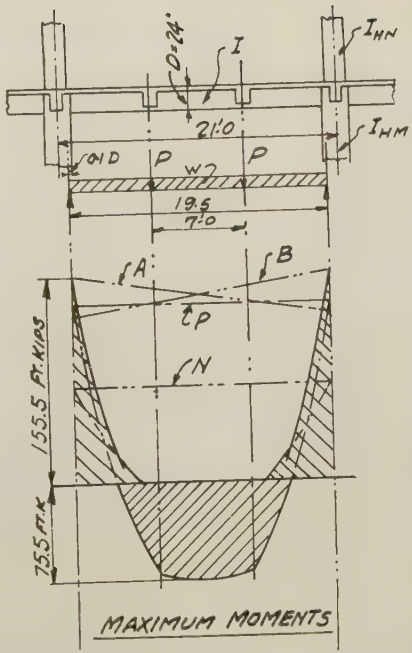
constants), this is not strictly correct, as will be noted from the following: For loading as shown Fig. 9A, typical interior bays, alternately loaded and unloaded (condition of maximum positive moments), and floors over and under uniformly loaded and therefore without effect on columns, the column moment is 0 at a distance B from top, and $H' = \frac{H}{2}\left[3 - \frac{H}{H-B}\right]$

This is the condition as expressed in values for H'/H in Table 2. The condition of loading as shown in Fig. 9B assumes that all stories are alternately loaded and unloaded. The point of contraflexure of the columns occurs at the joint and $B = 0$, $H' = H$. This would give higher values for the positive moments, but as this condition of loading is very unlikely, it is not generally adopted in commercial design.

Points of contraflexure as shown in Fig. 9C would occur when bays are correspondingly loaded and unloaded throughout. $B = H/2$, $H' = \frac{1}{2}H$. This condition of loading is of no practical value except that a similar deformation of columns takes place for dead loads (except for typical interior bays where there is no column bending), making $H' = \frac{1}{2}H$ more accurate in determination of the dead load coefficient. For comparison of results from this method with results derived from the usual application of the slope deflection method, assuming columns fixed at adjacent floors, note that $B = \frac{1}{3}H$, or $H' = \frac{3}{4}H$.

For the practical application of this method, the reader is referred to examples in Figs. 10 and 11.

Side Movements. It should be noted that all floors are assumed to be held in position so that no form of loading can cause any horizontal movement. Dead loads have no such effect, nor does any symmetrical form of live loads. It is only loading not symmetrical about the centre line of the complete frame system that will have any effect, and for many columns in the frame the effect is negligible. For building construction it is fair to assume that not all bays will cause movement in one direction and that the floor will act to even out these tendencies so that they can be neglected.



I_{HM} = 28000, H_M = 10'9", H_M' = 0.81 × 10.75 = 8.7, $\frac{I_H}{H}$ = 3200 } 5000
I_{HN} = 13000, H_N = 9'0", H_N' = 0.81 × 9.0 = 7.2, " = 1800 } AVE 2500
I = 25000, L = 21'0 $\frac{I_H}{H'} \cdot \frac{L}{I} = 2500 \cdot \frac{21}{25000} = 2.1$

USE DIAGRAM "TYPICAL BAY"

DEAD LOADS, P_D = 10 K
BEAM W_D = 0.25 K } 0.65 K
WALL W_D = 0.40 K

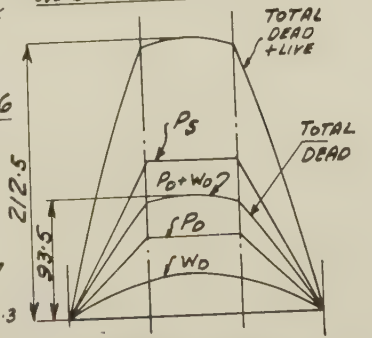
$K_D = \frac{1}{3} \times 10 \times 19.5 = 65$ FT.K
$+ \frac{1}{8} \times 0.65 \times 19.5^2 = 31$ " } 96

LIVE LOADS P_S = 19 K

$K_S = \frac{1}{3} \times 19 \times 19.5 = 123$

$M_{DL} = M_{DR} = 0.67 \times 96 = 64.5$

$M_{PL} = M_{PR} = 0.59 \times 123 = 72.5 + 64.5$ } 137
$M_{NL} = M_{NR} = 0.08 \times 123 = 9.8 + 64.5$ } 74.3
$M_{AL} = M_{BR} = 0.74 \times 123 = 91.0 + 64.5$ } 155.5
$M_{BL} = M_{AR} = 0.51 \times 123 = 62.5 + 64.5$ } 127.5

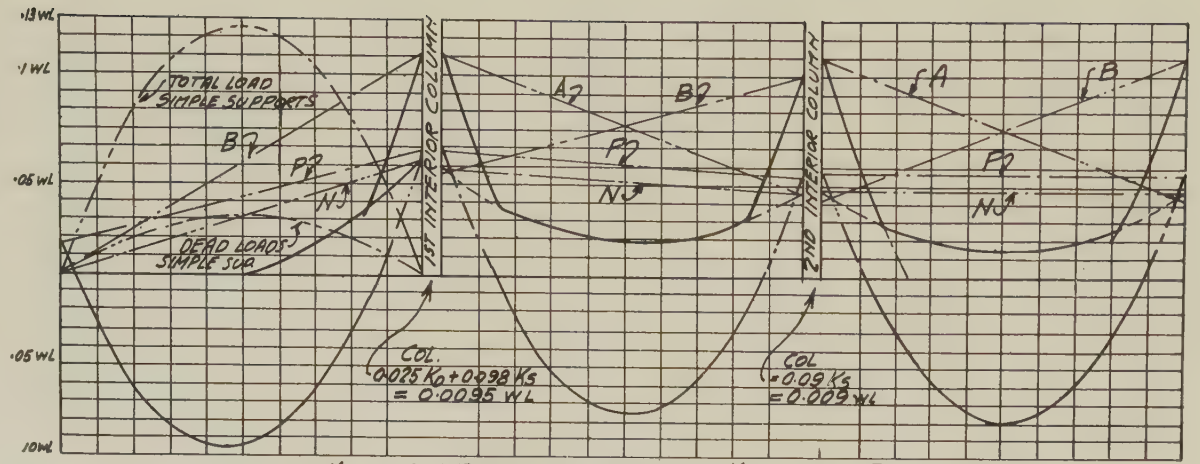


MOMENTS FOR SIMPLE BEAM SUPPORT

MAX SHEAR 10 + 19 + 0.65 × $\frac{19.6}{2}$ = 35.3 } 36.7 K
+ (155.5 − 127.5) × $\frac{1}{19.5}$ = 1.4

COL. BENDING $D_C \times K_D + S_C \times K_S = 0 + 0.5 \times 123 = 61.5$ FT.K
COL. UNDER. $\frac{3.2}{5.0} \times 61.5 = 39.3$ FT.K COL OVER $\frac{1.8}{5.0} \times 61.5 = 22.2$ FT.K

FIG. 10—TYPICAL EXAMPLE. BEAM MOMENTS



$K_D = 0.25 \times 0.125\ WL = 0.0312\ WL$; $K_S = 0.75 \times 0.125\ WL = 0.0936\ WL$

$M_{DL} = 0.098\ K_D = 0.00306\ WL$
$M_{DR} = 0.829\ K_D = 0.0259\ WL$
$M_{PL} = 0.129\ K_S + 0.00306\ WL = 0.015\ WL$
$M_{PR} = 0.447\ K_S + 0.0259\ WL = 0.068\ WL$
$M_{NL} = +0.032\ K_S + 0.00306\ WL = \sim 0$
$M_{NR} = 0.382\ K_S + 0.0259\ WL = 0.062\ WL$
$M_{AL} = 0.129\ K_S + 0.00306\ WL = 0.015\ WL$
$M_{AR} = 0.447\ K_S + 0.0259\ WL = 0.068\ WL$
$M_{BL} = 0.080\ K_S + 0.00306\ WL = 0.002\ WL$
$M_{BR} = 0.924\ K_S + 0.0259\ WL = 0.112\ WL$

$M_{DL} = 0.804\ K_D = 0.0251\ WL$
$M_{DR} = 0.634\ K_D = 0.0198\ WL$
$M_{PL} = 0.455\ K_S + 0.0251\ WL = 0.068\ WL$
$M_{PR} = 0.358\ K_S + 0.0198\ WL = 0.053\ WL$
$M_{NL} = 0.349\ K_S + 0.0251\ WL = 0.058\ WL$
$M_{NR} = 0.276\ K_S + 0.0198\ WL = 0.046\ WL$
$M_{AL} = 0.918\ K_S + 0.0251\ WL = 0.111\ WL$
$M_{AR} = 0.244\ K_S + 0.0198\ WL = 0.043\ WL$
$M_{BL} = 0.315\ K_S + 0.0251\ WL = 0.055\ WL$
$M_{BR} = 0.862\ K_S + 0.0198\ WL = 0.100\ WL$

$M_{DL} = M_{DR} = 0.641\ K_D = 0.020\ WL$
$M_{PL} = M_{PR} = 0.366\ K_S + 0.020 = 0.0543\ WL$
$M_{NL} = M_{NR} = 0.275\ K_S + 0.020 = 0.045\ WL$
$M_{AL} = M_{BR} = 0.867\ K_S + 0.020 = 0.101\ WL$
$M_{AR} = M_{BL} = 0.222\ K_S + 0.020 = 0.041\ WL$

5 SPANS $\frac{I}{H'} = 0.1\frac{I}{L}$
LIVE LOAD = 3 TIMES DEADLOAD

FIG. 11

CONCLUSION

All constants in Table 3 have been figured following the method originally published by A. Strassner in his book "Neuere Methoden zur Statik der Rahmentragwerke." This method eliminates long equations, and any error is very soon discovered.

The writer wishes to acknowledge helpful suggestions received from M. P. van Buren of J. Di Stasio & Co., New York City.

APPENDIX 1

The Strassner method is a continuation of and improvement on the Ritter method. Both methods are based on Mohrs principle "that the ordinates of the elactic line are the bending moments of the beam from a loading at each point equivalent to the moment area divided by $E.\ I$."

Strassner derives two points, J and K, for each member of the frame system, J near the left support and K near the right support. The location of these points expresses the combined elastic value (stiffness) of all of the structure that lies respectively at the left of J and at the right of K. In the determination of J enter the values of E, I and L for all members, beams and columns, of the left part of the structure, and for K, in a similar manner, values for the right part.

As the basic frame in this paper has columns hinged at opposite ends, the above mentioned fixed points need only be determined for beams. The general equations are shown in Fig. 12. For the various stiffnesses investigated they are for J points:

$I_H/H^1 \cdot L/I$	a_1	a_2	a_3	a_4	a_5	
0	0.0000	0.2000	0.2105	0.2112	0.2113	
0.10	0.0769	0.2170	0.2237	0.2241	0.2241	
0.40	0.1818	0.2480	0.2500	0.2500	0.2500	$\times L$
1.00	0.2500	0.2758	0.2763	0.2764	0.2764	
10	0.3226	0.3232	0.3232	0.3232	0.3232	

The same values apply to K points, but starting at the extreme right support.

The support moments are figured independently for loading at any one span. A load on span CD, Fig 12, created moments at the J and K points, the values of which are directly proportional to the moments at the support. See Fig. 13.

$$M_J = -\frac{a}{L} \cdot \phi_C \cdot \frac{6 \cdot E \cdot I}{L}; \quad M_K = -\frac{b}{L} \cdot \phi_D \cdot \frac{6 \cdot E \cdot I}{L},$$

where ϕ_C and ϕ_D are the angular deflections from the simply supported load only, at supports C and D respectively. Referring to Table 1 in this paper,

$$\phi_C = \phi_D = \tfrac{1}{3} \cdot K \cdot \frac{L}{E \cdot I} \quad \text{or}$$

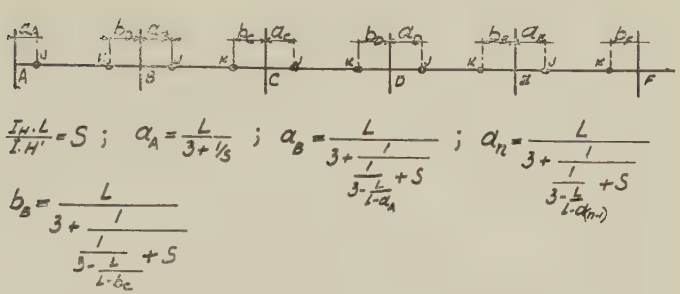


FIG. 12—GENERAL EQUATIONS, STRASSNER METHOD

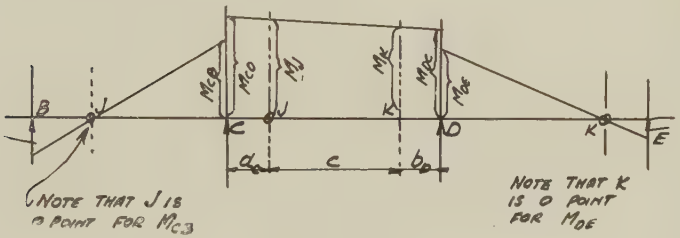


FIG. 13

$$M_C = -\frac{a}{L} \cdot 2K \cdot \frac{L - 2b}{c} \quad ; \quad M_D = -\frac{b}{L} \cdot 2K \cdot \frac{L - 2a}{c} .$$

Having determined the support moments from loading in any one span, it remains to distribute them between adjacent beams and columns.

From span CD in direction CB

$$M_{CB} = M_{CD} \cdot \frac{1}{1 + \frac{L}{H'} \cdot \frac{I_H}{I}\left[3 - \frac{L}{L - a_B}\right]}$$

From span CD in directon DE

$$M_{DE} = M_{DC} \cdot \frac{1}{1 + \frac{L}{H'} \cdot \frac{I_H}{I}\left[3 - \frac{L}{L - b_E}\right]}$$

The differences between the moments at adjacent beam supports are distributed on columns over and under in direct relation to their stiffnesses.

The above mentioned values are, for beams right to left:

$I_H/H^1 \cdot L/I$	*BC* to *BA*	*CD* to *CB*	*DE* to *DC*	*EF* to *ED*
0	1.0000	1.0000	1.0000	1.0000
0.10	0.8392	0.8530	0.8538	0.8538
0.40	0.5844	0.5994	0.6000	0.6000
1.00	0.3750	0.3818	0.3820	0.3820
10	0.0616	0.0616	0.0616	0.0616

Values left to right are determined in a similar manner

APPENDIX 2

The support moments from a concentrated load in any span can be determined (see Fig. 14) as follows:

$$M_L = -\frac{P \cdot X \cdot (L - X)}{L} \cdot \frac{a}{L} \cdot \frac{2L - X - 3b}{C}$$

$$M_R = -\frac{P \cdot X \cdot (L - X)}{L} \cdot \frac{b}{L} \cdot \frac{L + X - 3a}{C}$$

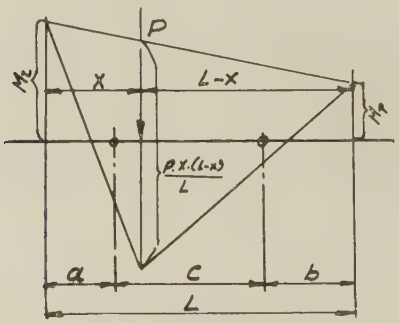


FIG. 14—SUPPORT MOMENTS FOR CONCENTRATED LOAD

These formulas can be used to make allowance for any variation to the assumed symmetrical loading, as the fixed points and distribution of moments can be interpolated from the above tables using the same scale as for the diagrams in this paper.

Readers are referred to the JOURNAL *for April 1931 for discussion which may develop. Such discussion should reach the Secretary by March 1, 1931.*

Discussion of Paper by Willis A. Slater and Inge Lyse

"Compressive Strength of Concrete in Flexure—As Determined from Tests of Reinforced Beams"*

BY FRITZ EMPERGER†

The relation between the compressive strength of concrete in flexure and that in direct compression has recently received increased interest as the methods of designing concrete of a given strength have become better known, and thereby it has become advisable to use lower factors of safety.

In this connection it is not sufficient to have a value like the cube strength which never appears alone in a structure. We have on one side to find the correct column strength, and on the other side the compressive strength in beams. With respect to the columns it is considered established that the column strength is 80 per cent of the cube strength. The question involved is, therefore, to find the multiplication factor for flexural compressive strength.

As a result of my proposal in *Beton und Eisen*, 1903, I tried in 1906, to introduce the use of heavily reinforced concrete beams as control specimens in the construction field, and was able to establish its use at a number of places, as for instance, in the construction of the Building for the Department of War in Vienna. The beam used had a cross-section 70 by 200 mm. (2¾ by 4 in.). The cross-section was later changed to 70 by 96 mm. on account of supporting the steel directly on the form. The loading arrangement was selected such that by the use of a value of n equal to 15, the flexural compressive strength is equal to $P/3$ without further computations.

An investigation by the Austrian Committee for Reinforced Concrete was undertaken in the same year for the purpose of

*A. C. I. Journal, June 1930; *Proceedings*, Vol. 23, p. 831.
†Vienna, Austria.

studying the proper shape and multiplication factor for such a test beam. The given dimensions resulted from the effort at reducing both dimensions and load to such a degree that the beam would be easily handled and could be tested without a testing machine. The studies by the Austrian Committee for Reinforced Concrete included more than 400 beams and a similar number of cubes. The first tests were carried out with fixed beams[1] and later a special series of tests was made.

The voluminous report[2] of these tests was not published until 1916 on account of the war. This discusses different concrete mixtures, beam cross-sections, and reinforcements. The reinforcement used in the beams amounted to 2.02, 4.04, 4.24, 4.97, and 5.83 per cent. Beams of different widths and depths were tested which were constructed according to field methods, as these tests were primarily intended for use on the job. In spite of this fact, the beams showed almost the same variation from the average as that obtained by the tests of the carefully made cubes. The maximum variations for the beams were +19.2 per cent and —16.8 per cent, and for the cubes +13 per cent and —18.2 per cent. The strength of the concrete used was as high as 420 kg. per sq. cm. (6000 lb. per sq. in.). The reinforcement was ordinary steel with a yield point stress of 2400 kg. per sq. cm. (34,000 lb. per sq. in.). It was established that for the ordinary strengths used in constructions, a reinforcement of 4 per cent was sufficient for the concrete in the beam. A width of 7 cm. was considered sufficient to accommodate the maximum size of aggregates used in reinforced concrete construction, since the use of these beams was not considered for large size aggregates.

In the course of progress of these tests, the German Committee for Reinforced Concrete also raised the same question, and received accurate information from us regarding the progress of the work.

The German tests are reported by Bach in Heft 19 of the Committee's publications. The beams were of larger width and had shear reinforcement, and were loaded at the center. As the

[1] "Berichte des Oesterreichischen Eisenbetonausschusses" Heft 4 by Dr. Fritz Emperger.
[2] "Berichte des Oesterreichischen Eisenbetonausschusses" Heft 6 by Hofrat Nähr.

latter method gives only the results for a narrow element across the beam it was soon abandoned.

The publication had the result that many doubts were raised in the German technical literature, while in Austria it had already been decided to adopt the beam test in the specifications. A second publication by Dr. Petry in Heft 50 of the reports from the German Committee for Reinforced Concrete, met the various objections and also showed that the cubes could not be made reliable on the job while the test beam could. Finally, the test beams have been included in the German specifications of 1926, and in the later specifications for concrete control, and at the same time a handy testing machine for the observations of maximum loads has been developed.

Before we discuss the important relationship between cubes and beams, we will study some American tests. The first publication was by Slater and Zipprodt in the *Proceedings* of the American Concrete Institute for 1920,[3] and then by Slater and Lyse in the American Concrete Institute's JOURNAL for June, 1930. It should be first mentioned, that the American engineers have changed from the use of the cube. It was first established in Germany that the cube strength is not even present in the columns and that the prism strength or actual strength of the concrete is about 80 per cent of the cube strength. No conclusion, however, has been drawn from these results, and the cube form has remained an artificial measure of the strength and other properties of the concrete. In North America, however, a cylindrical form of 15 cm. diameter and 30 cm. height has been adopted and this specimen gives numerous advantages over the cubes. It is always easy to make the cylinder mold from a sheet of galvanized iron, and glass plates will serve as end plates. It is of special importance to have a uniform method of filling and this cannot be secured in the corners of the cubes. The same difficulty is present in the capping of the cubes which might produce uneven load distribution. It seems probable that the cylinder sooner or later will be adopted also in Europe. The 30-cm. cubes give 13 per cent higher strength than the cylinder, the 20-cm. cubes give

[3] "Compressive Strength of Concrete in Flexure," by W. A. Slater and R. R. Zipprodt, A. C. I. *Proceedings*, Vol. 16, p. 120.

20 per cent higher strength, and the cylinder strength can be considered equal to the columns. (In order not to confuse the reader, we will use the 20-cm. cube strength throughout the rest of this discussion.)

It is of especial interest to note that the American tests included a further form of flexural strength in its program. This corresponded to the important transition form between pure compression and pure bending by eccentric load on compression specimens, which results in combined bending and direct compression. We see from these tests that the actual distribution of stresses does not correspond completely to that given by the assumption of Hooke's law, and that the error is due to this assumption. The eccentric tests have shown that at maximum load the compressive strength is 20 per cent higher than the cylinder strength, even though the relation between base and height was the same in both cases. In this case the load does not correspond to the ordinary column strength, but to the 20 per cent higher cube strength.

Further studies by Slater and Lyse bring forth that the ratio decreases as the strength of the concrete increases, so that the leaner concrete shows a relatively higher flexural strength than the richer concrete. This is a new proof of the fact that this concrete contains strength properties which should not be underestimated and which is very useful where the structure is expected to deform under secondary stresses.

When we consider the question of multiplication factor it must be remembered that the Austrian specifications assume 1.33, and the German tests show 1.70 and higher. A superficial reader will find a great difference between the results for the German and the Austrian tests. As the Austrian tests and original reports are very little known, it is quite regrettable that the German writers have not taken notice of them. The fact is that the German figure is based on the 30-cm. cube and the beam loaded at the center. According to the limited tests in Heft 19 "Deutsche Ausschuss für Eisenbeton" we find that this arrangement gives 9 per cent too high a value. We have, therefore, first to subtract from 1.7, 9 per cent due to the center load, and then 7 per cent due to the use of the 30-cm. cube when we want to obtain the relation for the 20-cm. cube. We find then, that the

German tests give a result of 1.45 as multiplication factor, or only slightly different from the specified Austrian value. The Austrian value did not correspond to the tests made, but was purposely chosen somewhat lower. The average of the tests in both countries gave approximately the same value. The question arises why the American tests give so much higher values. These tests may not be convincing because they were obtained on a small number of specimens, as also some of the Austrian tests showed higher values in some of the groups of tests. In order to arrive at a useful average, a much larger number of tests and also tests including different aggregates should be made, as the American tests only included one kind of aggregate. In order to obtain a complete view of the situation and summarize the differences obtained, it may be said that the multiplication factor given by the Austrian specifications is too low and instead of 1.33 the number 1.5 corresponds more closely to the actual relation. Otherwise a too high cube strength would be derived from the control beam. One thing should not be overlooked, and that is that the cube strength should not be considered fixed and constant, and the flexural strength liable to large variations. The situation is more clear when we disregard the imagined strength and judge the case by the deformation at failure, and keep in view the fact that the failure neither by cubes nor by beams, is caused by compression, but by the shear stresses which cause the deformation at failure. We will then see that the deformation at failure depends upon the whole set of conditions, that is, from the dimensions of the specimens to the quality of the concrete, and that the ratio is subject to large variations, and cannot be given a fixed value. When, however, we use such a ratio it is necessary to fix the limits which will be within the given factor of safety and not let the selection of the ratio depend on anyone's judgment. The increase in compressive strength obtained by eccentric loading is important when the eccentric load has been neglected in the design of structure. In the building of arch bridges where this should be done with great accuracy, this increase is found to amount to 20 to 50 per cent and increases with the eccentricity.

How sensitive the beam strength is with respect to all kinds of effects is brought forth in my report in *Forscherheft*, 14, 1911,

which contains the use of a strength thermometer by which it was possible to conclude what temperature was present at the time of set of the concrete from the strength results of the test beams.

AUTHORS' CLOSURE

The authors are glad that Dr. Emperger has given some historical information on the Austrian and German investigations designed to determine the compressive strength of concrete in flexure and of the use of small heavily reinforced beams as control specimens. Certain advantages of this form of control specimen are recognized. Its principal disadvantage probably lies in the fact that it is more difficult and expensive to make and test this control beam than it is to make and test a small cylinder or a cube. Due to this fact it would not be feasible to provide control tests from as many batches of concrete as may be sampled for a given cost by means of small cylinders or cubes.

Dr. Emperger indicates that the Austrian values are considerably lower than those given in this paper. Unfortunately we have not been able to compare his results with ours. Finality is not claimed for the results of our tests. Nevertheless, every reasonable effort was made to keep the tests free from effects which would vitiate the results. It is recognized that much more ground could be covered in an investigation which included 400 instead of 36 beams. However, three beams of each kind were made and the close agreement between results for individual beams should be evidence that the work was carefully done. In the absence of any suggestion as to any features in which the methods used were defective, the authors are inclined to believe that the results may safely be applied within the field which the investigation covered.

It is quite possible that if long-time loads had been applied to the beams, lower strengths would have resulted. However, the same is true regarding the cylinder tests, and there is no certainty that the ratio of the compressive strength in flexure to that in direct compression would have been any lower than those found, if all the tests had been made under long-time loading. So far as the authors know, this is the most important feature not covered in the investigation.

An effort was made to review all the previous literature on the subject under investigation, and it is regretted that the tests carried out by Dr. Emperger were not found in the search that was made. His discussion of this paper is highly appreciated.

The authors wish to correct an error which has been pointed out in Table 7 of the paper under discussion. The computed stresses at failure, based upon the observed values, of n as given in Column 6 and the resulting beam-cylinder ratios as given in Column 9 of Table 7 are both somewhat too small. The correct values are:

Column 6	*Column 9*
2340	1.68
3130	1.12
4250	1.04
5050	1.05
5680	.99
3670	1.42
4140	1.00
3550	1.20
3460	1.25
3570	1.23
3080	1.09
4120	1.08

The error is deeply regretted, but fortunately it is not of sufficient magnitude to affect any conclusions in the paper. Even the curves in Figures 21 and 22 are not affected appreciably by the error.

Properties of Mass Concrete

Report of Committee 108

A Summary of Existing Data With A Suggested Program Of Investigation

BY RAYMOND E. DAVIS*, CHAIRMAN, AND G. E. TROXELL†

The hardening of portland cement concrete is a chemical process, and the hydration of the cement, being an exothermic reaction, is accompanied by a liberation of heat. When water is available, it appears that the rapidity of hydration depends upon the chemical composition of the cement, the fineness of the cement particles, and the temperature of the concrete of which the cement is a component part. Other things being equal, the finer the cement and the higher the temperature, the more rapidly does hardening take place. For most portland cements, hydration practically ceases for temperatures approaching the freezing point of water.

The total heat liberated by a given amount of cement from the time of beginning of hydration until hydration has been completed, can not readily be determined, but perhaps within ordinary limits of grinding may not differ as greatly with the chemical composition of the cement as is sometimes supposed, even though the rate of heat generation during the early stages of hardening may be very much greater for one cement than for another.

Any concrete, the temperature of which at the time of mixing is not above that of the surrounding medium, will undergo an increase in temperature, the heat generated by the hydration process being absorbed by the aggregates and uncombined water

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and gradually being dissipated by radiation until its temperature corresponds with that of the surrounding medium. At the beginning, temperatures increase rapidly, as does the hydration process. Then, as hydration is retarded, the rate of heat generation becomes less, until when rate of heat generation equals rate of heat dissipation, the maximum temperature is reached. Thereafter, as the rate of hydration gradually declines, the temperature decreases until that of the concrete reaches that of the surrounding medium.

It might be expected, then, that the maximum temperature attained by concrete due to chemical heat and also the time required to reach this maximum temperature would be influenced by (1) chemical composition of cement, (2) fineness of grinding, (3) character of aggregate, (4) richness of mix, (5) water-cement ratio, (6) initial temperature of the concrete, (7) temperature and character of the surrounding medium, and (8) the size and shape of the concrete mass. Thus it appears that there are at least eight factors, a variation in anyone of which may influence to greater or less degree the maximum temperature.

It is a matter of common experience that with our normal portland cements and normal mixes, the temperature rise in structures of thin section is too small to be of consequence. But it is also an observed fact that in concrete of massive proportions such as large dams, heavy foundations, thick arches, and the like, there may be a very considerable increase in temperature of the concrete subsequent to its placement, this increase sometimes amounting to more than 100 degrees F. and the interval during which this increase takes place sometimes being as long as a month or more. Because of the possible effect that these large changes in temperature may have, not only upon the properties of the material itself, but also upon the intensity and distribution of stresses and strains within the structural mass, it is of very great importance that the facts concerning the behavior of concrete in large masses should be known.

What follows is a summary of available information concerning the properties of mass concrete obtained for the most part from published writings, but to a considerable extent from unpublished data. An examination will show that beyond records of tem-

perature in a considerable number of structures, and some rather incomplete laboratory tests to correlate temperature and strength, very little is known of the properties of mass concrete. The observations of massive structures all indicate that the temperature rise is rapid and the temperature decline is gradual, in some cases extending over several years. While little quantitative data are available, it appears that in certain structures the decrease in temperature has been accomplished by cracking, indicating large tensile strains, and that in a general way the width and number of cracks increase with time. There are no data to show whether or not there is still further decrease in volume, after normal temperature has been attained, due to a gradual giving up of moisture within the mass which moisture is not employed in the hydration process; but it seems not unreasonable to suppose that mass concrete exposed to drying conditions may continue to give up moisture and gradually to shrink for many years after it has arrived at a state of practical temperature equilibrium, just as at a more rapid rate does the concrete in structures of thin section.

From such laboratory tests as are available it appears that high setting temperatures in mass concrete lead to high early compressive strengths but that ultimately the strength of concrete cured at normal temperatures is in excess of that which has undergone the more elevated temperatures within the mass. There is also evidence that when the temperatures are considerably above normal, the concrete within the mass may reach its maximum strength within a few days and thereafter may suffer a slight retrogression in strength, but tests are of too short a duration to demonstrate whether this retrogression is not perhaps later followed by a second increase in strength which may equal or exceed the amount of the retrogression. There is no information concerning the elastic or plastic properties of concrete subject to conditions obtained within the mass, nor concerning shrinkage strains.

EARLY TESTS

Results obtained from some investigations to determine the temperature rise due to the setting of cement follow:

Tests reported by J. B. Johnson on the rise in temperature at the center of a mass of portland cement concrete 11.5 ft. thick, show that the temperature rose 85.5 degrees F. in 4 days and reached its maximum increase of 93.6 degrees F. in 7 days, after which it fell off slowly. Data on the mix used are not stated.

The Watertown Arsenal made 12-in. cement cubes using 27 different brands. These specimens were in wooden forms with their tops exposed to the air. For these cubes, having an average initial temperature of 77 degrees F., the maximum temperature was 218 degrees F., showing a rise of 141 degrees in 13 hours. The lowest maximum temperature for any of the portland cements was 117 degrees F., a rise of 40 degrees.

The materials testing laboratory at Lehigh University made an 8-in. cube of 1:3 mortar, which rose from 70 to 160 degrees F. in 18 hours while the laboratory temperature was constant at 70 degrees F.

Tests on 6-in. neat cement cubes and on concrete as actually placed in construction work were made by the Universal Portland Cement Company at Buffington, Indiana. The 6-in. cubes rose from 80 degrees F. to 130 degrees F., a rise of 50 degrees in 8 hours, while the concrete rose from 70 to 110 degrees F., a rise of 40 degrees F. in 40 hours.

BOONTON DAM TESTS

Thaddeus Merriman,[1] one of the early investigators of temperatures developed in mass concrete, carried on tests during the construction of the Boonton dam in New Jersey in 1903. The total length of this dam was 3100 ft. and the maximum height of the full masonry section was 114 ft. Atlas portland cement was used exclusively in its construction. Although the downstream face was faced with ashlar masonry, the interior was of cyclopean concrete composed roughly of 50 per cent of concrete of a 1:3:6 mix and 50 per cent of large stones. All stone used was a syenitic gneiss and the sand was of glacial origin.

As work on the Boonton dam progressed, eleven thermophone coils were placed in the concrete at different elevations and at various distances from the two faces of the dam, but all in one central section. These thermophones were designed to be sensitive and correct to within 1 degree F.

[1]See Bibliography page 418 for all references.

The observed temperature record for thermophone No. 11, which was 8.5 ft. from either face and 11 ft. below the top of the dam, is typical of that for the other positions. The coil was set into the dam on June 24, 1904, and was immediately covered with about 12 in. of concrete which was at a temperature of 74.5 degrees F. Within the next 16 hours, it attained a temperature of about 90 degrees F. Seven days later, when the coil indicated a temperature of 77 degrees F., more concrete was placed above it, so that within 14 days, it was buried to a total depth of 11 ft. A second rise in temperature from 77 degrees to 93 degrees F. occurred in 13 days. This second temperature rise was largely due to the heat from the masonry placed above it, and consequently it can be said that the temperature of the masonry placed above coil No. 11, and insulated from it by 12 in. of concrete already set, must have reached a temperature higher than 93 degrees F.

The concrete in this structure was deposited in thin layers and therefore the heat was quickly dissipated. Merriman expresses the opinion that a temperature rise of 40 degrees F., could have been anticipated had the heat losses been reduced by depositing the concrete in deeper layers.

In the Boonton dam, about one year elapsed before the masonry returned to normal temperature. The Boonton dam records also indicate that minimum and maximum temperatures near the center of the dam due to seasonal temperature variations are reached approximately 30 days after the occurrence of the atmospheric minimum and maximum, respectively, and moreover, the "lag" is proportional in some degree to the distance from the nearest face of the dam.

R. H. Brown[2] reports tests conducted during the construction of canal walls for power development on the Kennebec River in 1909, at Madison, Maine. The concrete was a 1:3:5 mix, placed at a temperature of 70 degrees F. Recording thermometers placed inside a 3-in. pipe at the center of the thick walls (thickness not stated) registered a maximum temperature of 119 degrees F., reached after 8 days, when the depth of concrete over the thermometer was 26 ft. The temperature dropped to 98 degrees F. when the concrete was 20 days old. During these

observations, the outside air temperature averaged about 20 degrees F.

PANAMA CANAL LOCKS

Tests made to determine the setting temperatures in the Panama Canal Locks[3] show that a maximum temperature rise from 106 degrees to 130 degrees F. was attained 12 ft. from the surface. The maximum temperature occurred about 10 days after placing the concrete. Temperatures were observed by means of electric resistance thermometers imbedded in the side walls of the locks. The concrete was a 1:3:6 mix with some large plums.

DES MOINES ARCH BRIDGE

The concrete placed during construction of the Walnut Street arch bridge[4 *and* 6] in Des Moines, Iowa, was tested for rise in temperature while setting. The bridge consists of six 68-ft. reinforced concrete arches, the arch rings being 16 in. thick at the crown and 6 ft. thick at the spring line. Nine electric resistance thermometers were used in the tests. The maximum temperature rise occurred at the spring line, 6 in. from the intrados, where the 1:2:4 mix of concrete, placed continuously during night and day, reached a temperature of 108 degrees F., showing a rise of 32 degrees F. within 44 hours after placing. The mean daily air temperature was about 70 degrees F.

CEMENT PASTE TESTS BY BEALS

To determine the maximum temperature attained, and also to determine the rate of evolution of heat by cement pastes, Louis N. Beals, Jr.,[5] in 1913 reported tests upon small samples. Preliminary tests made by placing 900 g. of cement plus 225 g. of water in a can surrounded by a larger second can with paper insulation between, showed temperature rises varying from 14 degrees F. to as much as 45 degrees F. for the different brands tested.

As the heat due to the chemical reaction is evolved rather slowly and extends over a considerable period of time, the heat losses from the two cans described above were quite large. A second series of tests was therefore run using 1- to 2-lb. samples of cement in a thermos bottle with a secondary jacket to prevent heat losses.

The total calories liberated per gram of cement, corrected for radiation, were computed and plotted against time. It was found that different cements showed an appreciable difference in the rate of heat liberation and in the total heat liberated. It was also established that cements liberating only small quantities of heat, later proved to be unsound.

Beals' graphs show that if the radiation is taken into account, the evolution of heat continues over a period of more than 100 hours. Curves drawn to show the rate of heat generation in calories liberated per gram of cement per minute indicate that "there is a sudden evolution at mixing, then a sharp minimum —probably close to the time of initial set—followed by a longer maximum reaching a peak at about 8 hours, which may mark a more intrinsic property of the cement than the conventional final set. Another minimum and maximum follows, the significance of which may be developed by further investigation." The rate curves differed quite radically in some instances, as some cements indicated very high rates of heat evolution for short periods while others liberated the heat more slowly but for longer intervals. The total heat liberated by these two types of cement may or may not be the same.

W. A. Hoyt[7] states that the amount of heat generated depends upon the size or mass of the concrete, the kind of cement used, the consistency of the mixture, and also upon the richness of the concrete. A 1:2:4 mixture of medium consistency placed at 70 degrees F., in masses 12 in. thick or more, should develop a temperature of 120 to 150 degrees F. within 24 to 36 hours.

ARROWROCK DAM

Temperature changes in the concrete of the Arrowrock Dam, a gravity arch structure 330 ft. high, near Boise, Idaho, are reported by Paul and Mayhew.[8] The major portion of the concrete consisted of 1 part sand cement (45 per cent pulverized granite with 55 per cent portland cement by weight), 2½ parts sand, 5 parts of pebbles, and 2¾ parts of cobbles. A richer face mix, which varied in thickness, consisted of a 1:2:4:2½ mix. The cement in this face mix was composed of 76 per cent sand cement and 24 per cent portland cement by volume.

Ten electrical resistance thermometers were embedded in the concrete, all in one vertical plane, but at various elevations; some thermometers were close to the face of the dam, whereas others were deeply embedded in the dam. The temperature indicator, consisting of a modified Wheatstone bridge calibrated to read temperature units instead of ohms, could be read to 0.1 degree F. The Leeds and Northrup Co., the manufacturers of the coils and indicator, claimed an accuracy of the apparatus as a whole to 0.5 degree F.

Concreting at the location proposed for a thermometer was usually discontinued temporarily and the concrete allowed to set for a few hours before placing the coil. In some cases, concreting was not carried on continuously, directly above the thermometers. The delay in concreting preparatory to placing a coil, the shallow depth of cover of 1.5 to 2 ft. placed in most cases, the use of sand cement and the leanness of the mix used, all worked toward the development of lower maximum temperatures than would have resulted otherwise. As the investigators stated, however, the purpose of their observations was to determine the temperatures produced under average placement conditions for large dams where lean mixes are ordinarily placed in about 2-ft. layers with not more than one or possibly two layers being placed during one shift.

The results of the observations by Paul and Mayhew are presented in Table 1. As a result of these tests which show a maximum temperature rise of 36.7 degrees F., it was concluded that large bodies of concrete deposited at a temperature of about 60 degrees F. develop a temperature of from 90 to 95 degrees F. within a period of 30 days and maintain nearly that temperature for several days thereafter. Also, the effect of the "setting heat" in concrete 20 ft. or more from an exposed face is felt for several years after the concrete is placed. Concrete near the center of the mass of a large dam probably retains some of this "setting heat" for 5 years or more after placing.

Paul and Mayhew also state that the effect of the temperature of the air in counteracting the heat generated in the concrete by chemical action, is illustrated by the fact that all the comparatively low maximum temperatures recorded by several of the

TABLE 1—TEMPERATURE TESTS ON ARROWROCK DAM (PAUL AND MAYHEW)

Thermometer No.	Location of Thermometer	Days Required for Temp. Rise	Concrete Cover at Max. Temp.	Av. Air Temp. During Rise, °F.	Air Temp. at Max. Concrete Temp. °F.	Concrete Temp. °F.		
						Initial	Maximum	Change
1	76 ft. from W. face; 8.5 ft. W. of block placed 5 months previously; 60 ft. above bedrock.	27	15	71	60	66.5	94.0	27.5
2	3 ft. from W. face	5	8	55	57	55.0	74.5	19.5
3	20 ft. from W. face	45	28	44	26	49.5	86.2	36.7
4	58 ft. from E. face; 3.5 ft. below temporary top for 7 mo.	15	3	36	50	44.0	64.6	20.6
5	3.5 ft. from E. face; temporary top for 7 months only 3 ft. lower	5	4	48	48	43.0	69.8	26.8
6	31 ft. from E. face	31 101	20 80	73	77	61.5 61.5	93.7 96.2	32.2 34.7
7	2 ft. from W. face*	1	1.5	70	69	74.0	77.6	3.6
8	10 ft. from W. face*	17	30	58	58	75.0	91.6	16.6
9	1 ft. from E. face*	1	2	70	69	73.5	78.0	4.5
10	9.5 ft. from each face	5	10	48	51	47.0	80.0	33.0

*Thermometers 7, 8 and 9 directly above concrete 12 days old and under only 1.5 to 2 ft. of concrete cover for first two weeks.

thermometers occurred at points where the distance to an exposed face was comparatively small, and that all the high maximum temperatures recorded by a few of the thermometers occurred at points where the distance to an exposed face was considerable, and over which concrete to a considerable depth was placed during a comparatively short period. Therefore, it appears probable that the speed of placing has fully as much influence on the temperature of mass concrete as either the season of placing or the richness of the concrete. The duration of the rise in temperature, due primarily to chemical action, is undoubtedly

largely dependent on the speed of placing, season of the year, consistency, brand of cement, and quantity of cement per cubic yard of concrete.

In a discussion of the results reported for the Arrowrock Dam,[8] W. M. Smith calls attention to the fact that the proportion of portland cement in the loose materials by volume was only 5 per cent in the bulk of the masonry and 7 per cent in the facing. In the ordinary 1:3:6 concrete mixture, there is 10 per cent of cement by loose volume, and in a 1:2:4 mixture, generally used for reinforced concrete, the cement amounts to over 14 per cent. The statement is then made that had the cement component been higher in the concrete of the Arrowrock Dam, the setting temperatures would have been materially higher.

ASHOKAN RESERVOIR BRIDGE

The effect of a richer mix than that used in the Arrowrock dam is shown by tests on the spillway bridge at the Ashokan reservoir built by the Board of Water Supply of New York City. Several thermophones were placed at various points in the arch to record the temperature changes. Three were placed at the crown, five at a point about midway between the crown and the abutment, and three at the abutment. The arch was 3 ft. 4 in. thick at the crown, 4 ft. 8 in. at the midway point, and 6 ft. 3 in. at the abutment. The temperature rise at the crown was 20 degrees F., the maximum temperature being 88 degrees F. At the midway point, the rise varied from 27 degrees to 40 degrees F., according to the distance from the surface, the maximum temperature varying from 82 to 95 degrees F. At the abutment the rise varied from 30 to 50 degrees F., according to the distance from the surface, and the maximum temperatures varied from 95 to 115 degrees F. In every case, the maximum temperature was reached within 4 days.

In another test to determine the maximum setting temperature of concrete conducted during the construction of a shaft by the Board of Water Supply of New York City, it was found that a temperature of 136 degrees F. was developed.

KENSICO DAM

A very comprehensive series of tests to determine the temperatures developed in massive concrete structures due to the chem-

ical activity of the cement were conducted during the construction of the Kensico dam at Valhalla, N. Y., in 1913 and were reported by George T. Seabury in a discussion of the paper by Paul and Mayhew.[8]

Kensico dam is a gravity structure 1850 ft. long on top; its maximum height is 307 ft. and its volume is about 900,000 cu. yd. It is of cyclopean masonry, 27 per cent of the mass being of large stone; the remainder is of concrete mixed in the proportions of 1:3:6, Atlas and Alsen brands of cement being used. The concrete was handled rapidly in 2-yd. buckets, so that the elapsed time between mixing and placing was very short, usually not more than 5 or 6 minutes.

The portion of the dam chosen for the installation of the 47 sensitive electrical resistance thermometers is near the center. The thermometers were placed in two vertical planes; one near a contraction joint, and the other midway between two contraction joints, or in the center of a section. In each vertical plane the thermometers were placed in corresponding positions, and at various elevations; near the up-stream and down-stream faces, at equal distances from both faces, and at the middle line between these faces.

The central portion of the dam containing the thermometers was built comparatively rapidly. In the fall of 1913, a height of 35 ft. was placed, and in that height 15 thermometers were installed. During the following summer, the masonry was carried to the top—a distance of 165 ft.—in six months, and this part contains the remaining 32 thermometers.

At the Arrowrock dam it was the practice generally to place the thermometers in concrete which had already partly set and had been exposed for a long enough period to lose, by radiation into the air, a portion of the heat that would be normally developed. The thermometers, too, were covered to only a relatively small depth in many cases, permitting more radiation to go on, and so obscure the record of the total heat generated. On the other hand, at Kensico dam, the thermometers were plunged into the plastic concrete the moment it was placed, and were buried as deeply and as rapidly as possible in order to determine, if possible, the maximum temperature to which such a structure would be subjected under rapid placement conditions.

The results of the tests show that for a number of the locations, well away from the face of the dam, the rise in temperature was about 40 degrees F. and this seemed to be unaffected by the temperature at which the concrete was placed. This resulted, in the summer, in a maximum temperature well above 100 degrees. The highest temperature observed—and that which it is believed is probably the maximum in any part of the structure—was 118.5 degrees F., but this maximum temperature was not reached until long after the concrete had attained its final set. When undisturbed by other conditions, the maximum temperatures were reached in about 15 days—some points obtained their maximum temperatures in 5 days and others not for 95 days.

The increase in temperature due to the setting of the cement in the concrete began as soon as the latter was placed, and increased at the rate of about 1 degree F. per hour, for 4 or 5 hours, when the rate gradually increased to from 8 to 10 degrees per hour, for an hour or so, and then suddenly dropped to about 0.5 degrees F. per hour for a considerable time. The period of most rapid heat development corresponds with that of the final set of cement under ordinary conditions. Many observations indicated a rise in temperature of from 25 to 30 degrees F. after this period. As a consequence of this, the investigators raised the question: What is the effect on the strength of a large mass of concrete rapidly placed, if in its interior, a temperature of from 25 to 30 degrees F. is developed after the cement has ob-obtained its final set and the masonry its solidified form? Is the increase in size which results from this heating sufficient to develop disruptive strains in the fresh masonry beyond its early strength?

The opinion is expressed by Seabury that large bodies of concrete deposited rapidly develop a temperature a definite amount greater than the initial temperature of the aggregate, probably dependent almost entirely, if deposited rapidly enough to minimize radiation, on the quantity of cement to the unit of the resulting mass, and that this maximum temperature may be even 135 or 140 degrees F.

Meager evidence showing how the brand of cement affects the quantity of heat generated was ascertained, as there was one place in the Kensico dam where two thermometers were placed

about 5 ft. apart and under conditions which were thought to be identical, with one exception. After each had been covered by about 2 ft. of concrete made with Atlas cement, additional cover over one thermometer was made using the Atlas cement, while Alsen cement was used in the mix over the other thermometer. Within the first 6 hours after being placed, the thermometer under the Alsen cement did not rise as high as the one which was entirely surrounded by Atlas cement, and which had no Alsen near it, but, subsequently, they reached almost exactly the same temperature. The evidence is, then, that the brand of cement appears to make some difference at a certain stage, but not ultimately. However, these two thermometers were too close together to give reliable results unaffected by the heat from the other mix.

Seabury differs with the statement of Paul and Mayhew that "it appears probable that the speed of placing has fully as much influence on the maximum temperature of mass concrete as either the season of placing or the richness of the concrete." It was clearly demonstrated on the work at Kensico that, provided the initial heat acquired in setting be not dissipated through a slight amount of cover, the rise in temperature is a constant for any given mix, irrespective of the season of placing. It appears that the speed of placing has a great deal to do with the rise of temperature so far as reducing it is concerned; but, on the other hand, it seems that with a sufficient amount of cover, the rise will be a constant for any given mix. From this it follows that the season of placing, or rather the initial temperature of the aggregates and mixing water, has much to do with the maximum temperature reached.

The investigators at Kensico concluded that the rise in temperature due primarily to the chemical action, is undoubtedly largely dependent on the speed of placing, consistency, brand of cement, and quantity of cement per cubic yard of masonry but that these factors do not necessarily affect the duration of the rise in temperature as stated by Paul and Mayhew.

EAST CANYON CREEK DAM

Rather meager temperature observations have been reported for the East Canyon Creek dam,[9] built in 1915, near Morgan,

Utah. This is a true arch structure, 190 ft. high, having a thickness of 26 ft. at the base and 5 ft. at the top.

The proportions of the concrete used in the body of the dam were 1 part cement; 3 parts sand, with some screenings up to ¾-in.: and 5 parts broken stone of 3-in. maximum size. Red Devil brand of cement was used, and the cement content ran 4.75 sacks per cubic yard of concrete.

The concrete was deposited in 2-ft. layers, working back and forth. The aggregates were often frozen, but hot water added at the mixer raised the temperature of the fresh concrete so that it was just above the freezing point as it came from the mixer.

The rise in temperature due to chemical changes in setting prevented damage from frost when the concrete in the form was covered at night. Canvas spread over the forms and salamanders burning coke, aided in preventing surface freezing; while the interior temperatures of the mass soon increased sufficiently to remove any frost occurring on the surface, if not too great in quantity.

The removal of 1½-in. pipes which had been placed in the contraction joints afforded a very favorable means of testing the rise in temperature in the concrete while setting. Tests of temperature by lowering a thermometer into the holes, indicated in every case a considerable rise, a typical test showing a temperature of 70 degrees F. at a depth of 15 ft. when the air was 14 degrees F., a difference of 56 degrees F. from the air temperature or a difference of about 35 degrees F. from the initial temperature of the fresh concrete.

TESTS BY YOSHIDA AT UNIVERSITY OF ILLINOIS

While carrying on studies of the cooling of fresh concrete in freezing weather at the University of Illinois, Yoshida[10] also made tests to determine the rise in temperature during the setting of concrete and mortar.

The mixes used in these tests were made with Universal portland cement. The 0 to ¼-in. sand and the ¼ to 1½-in. pebbles came from pits at Attica, Indiana. The concrete or mortar to be tested was poured into a mold of 1½-in. pine lumber large enough for a 12-in. cube. Two thermocouples were embedded

to measure the temperature, one at the center of the cube and the other at 3 in. from the center. The mold was put in the center of a large wooden box and the space around all six surfaces of the mold was filled with sawdust about 8 in. thick, to prevent the cooling of the specimen.

The initial temperature of the fresh concrete or mortar was the same as that of the laboratory. The two thermocouples showed about the same temperature, so the temperature at the center was used in the preparation of Table 2. The total rise in temperature, the rate of increase, and the time interval before the maximum temperature was reached were all variable, depending upon the proportions of the mix and the amount of mixing water.

TABLE 2—RISE IN TEMPERATURE DURING SETTING OF CONCRETE AND MORTAR (YOSHIDA)

Mix	Water-Cement Ratio	Consistency	Initial Temp.	Max. Temp.	Temp. Rise	Hours to Develop Max. Temp.
Mortar			°F.	°F.	°F.	
Neat	0.38	Medium	68	184	116	15
1:1	0.48	Medium	68	126	58	24
1:2	0.60	Medium	68	108	40	32
1:3	0.85	Medium	68	86	18	24
Concrete						
1:1:2	0.72	Medium	72	89	17	29
1:2:4	0.81	Dry	73	85	12	30
1:2:4	1.01	Medium	75	87	12	30
1:2:4	1.26	Wet	71	83	12	26
1:3:6	1.29	Medium	71	82	11	29

EMIGRANT CREEK DAM

During construction of the Emigrant Creek dam,[13] holes about 2 in. in diameter extending from the downstream face to within 3 ft. of the upstream face were prepared at three different levels.

The thickness of the dam at the lower, middle and upper levels, was 15.5, 10.5 and 5.5 ft. respectively.

Long wooden rods of a diameter slightly smaller than the openings were then inserted in the holes after several maximum registering mercurial thermometers had been embedded in the

rods. Felt washers were used to prevent circulation of air in the holes. Daily readings taken when setting was progressing most rapidly indicated temperatures as high as 132 degrees F. Data on the concrete mix are not stated.

CALDERWOOD DAM

Tests of the temperatures in the Calderwood dam, a thin-section-arch overflow dam 230 ft. high, indicated a temperature of 148 degrees F. at the center. The concrete mix was 1:3:4.5 with a cement content of 4.5 sacks per cu. yd. The water-cement ratio was about 0.85, producing a slump of 1½ in.

WILSON DAM

A very complete program of temperature observations conducted during the construction of Wilson dam on the Tennessee river has been reported by John W. Hall.[16]

This structure is of the gravity type and has a total length of about 4300 ft. The spillway section, where the temperature tests were made, is about 94 ft. high and was built in 1923 and 1924. The concrete contained 5 sacks of cement per cubic yard and the maximum size of the aggregate was 3½ in. This concrete was mixed at central mixing plants, then dumped in 4-cu. yd. buckets of the bottom dump type and transported by railroad to the place of deposition. Each layer of concrete averaged about 6 ft. in depth.

Electric resistance thermometers were embedded in the concrete of one section of the dam, to determine the maximum temperature developed in the process of curing, and also to determine the length of time required for concrete to lose the effect of heat caused by chemical action of the cement. The resistance coils were placed in two vertical planes, one on the center line of the block, and the other 6 ft. from the expansion joint. Twenty-four thermometers were placed in each of these sections. The degree of sensitivity permitted the temperatures to be read to ¼ degree F.

As the thermometers were 1 ft. below the top surface of the layer being placed, the air temperature affected the thermometer until the next layer of concrete was placed, after which the thermometer recorded the temperature developed in the mass where it was located.

Some of the general points of interest from this series of observations are:

Thermometers 1 ft. from the face showed a rise from 50 to 83 degrees F. in 5 days, and then a fall to 43 degrees in 30 days, after which the temperature increased and decreased with the air temperature.

Thermometers 2 ft. from the face showed a rise of temperature from 64 to 82 degrees F. in 24 hours, and then a fall to 73 degrees in 3 days, at which time a new layer of concrete was placed and the temperature rose again to 86.5 degrees F. in 5 days and then gradually fell till the temperature reached 56.5 degrees F. at the end of 2 months.

Thermometers 9 ft. from the face, showed a rise in temperature from 63 to 79 degrees F. in 2 days and then a fall to 70.5 degrees in 3 days, when a new layer of concrete was placed, excluding the effect of the air temperature. The temperature then rose to 100 degrees in 11 days, after which it gradually decreased, reaching 76 degrees at the end of 3 months.

Thermometers 20 ft. from the face showed a rise of temperature from 64 to 84 degrees F. in 24 hours, and then a fall to 76 degrees in 3 days, when a new layer of concrete was placed. The temperature then rose gradually to 101 degrees in 11 days, after which it fell to 78 degrees in 5 months.

Thermometers 26 ft. from the surface showed a rise from 47 to 113 degrees in 24 days, and then a fall to 81 degrees F. in 7 months.

Information obtained from these temperature records shows that in this mass concrete, the temperature near the face increased slightly for a short period of time, approximately 4 or 5 days, and then fell slowly for as much as 30 days, after which the rise and fall of the temperature was directly affected by the air temperature prevailing. At greater depths in the mass, the temperature rose to as high as 113 degrees F. over a period of approximately 3 or 4 weeks, and then gradually decreased for a period of as much as 8 or 9 months.

It is interesting to note that this concrete was placed in the winter season when the air temperature, varying from 10 to 70 degrees F., but usually from 40 to 50 degrees F., would tend

to decrease the maximum reached by the mass, and yet this maximum was 113 degrees F. Also, it was noted that concrete near an expansion joint, even though in contact with adjoining mass concrete, does not increase to the temperature reached by the center of the mass, although this difference is apparently not over 3 or 4 degrees F. The information shown by this record is that the central portion of any mass concrete is sometimes in a state of expansion while the external surfaces are in a state of contraction, and explains the cracks appearing in the surface of concrete which do not penetrate to the interior of the mass.

Because of the high continuing temperatures in concrete of considerable volume, it was concluded by Hall that, if cracks in permanent work are to be avoided, deposits of concrete should be limited to 4 ft. for any one layer. Such deposits should be allowed to set and cool at least 5 days before succeeding courses are placed. Also, the greatest horizontal dimension for any given deposit should be 25 ft.

STEVENSON CREEK TEST DAM

In 1926, Slater[17] conducted an elaborate series of tests for Engineering Foundation upon the Stevenson Creek Test Dam in California in order to determine the distribution of stresses in arch dams. Temperatures of the concrete were observed from the time the concrete was placed until several months later.

This test dam is 60 ft. high and has a thickness of 2 ft. for the upper 30 ft. but below this the thickness increases until at the bottom it becomes 7.5 ft.

The concrete mix consisted of 1 part Colton cement, to 3 parts fine crushed granite aggregate varying from 0 to $\frac{3}{8}$ in., to 2 parts crushed granite varying from $\frac{3}{8}$ to $1\frac{1}{2}$ in., these proportions being by weight. The water-cement ratio varied from 0.87 to 1.03 and the consistency varied from a 2-in. to 7-in. slump.

In placing the concrete, a lift of 15 in. was deposited from end to end of the dam. The next lift was started at such time that at all points the concrete would have at least an hour in order for some set to take place between lifts. Ordinarily, one day's pouring consisted of four such lifts, or 5 ft. total. In general, about four days elapsed between pourings.

The setting temperatures of the concrete were measured by means of 140 electric resistance thermometers set at various points in the mass; some were close to the forms while others were deep seated in the concrete; some were at the middle vertical sections while others were at the abutments.

On the center line of the dam and 4 ft. above the base, where the thickness of concrete was 6 ft. 2 in., the temperature in the center of the mass rose from 73 degrees F. to a maximum of 116.5 degrees F. in about two days. From then on it gradually decreased, becoming constant at about 63 degrees F. after 3 weeks. At 2 in. from the face, the temperature rose from 72 degrees F. to a maximum of 95 degrees F. within one day, and from then on decreased at about the same rate as at the center, reaching a steady temperature of about 52 degrees F. after 2 weeks. The average air temperature was about 72 degrees F. when the maximum temperatures were reached. The minimum daily air temperatures averaged about 55 degrees F., and the maximum daily air temperatures averaged about 81 degrees F. during the first 3 weeks while the above mentioned tests were in progress.

MINNEAPOLIS BRIDGE

Jakkula[18] reports tests made in 1927 to determine the setting temperatures in the concrete piers of the Cedar Avenue bridge in Minneapolis. The concrete in this structure was made of portland cement, washed sand and trap rock, and the average field mix was 1:2.9:4.6. An inundator was used to measure the quantity of sand and to eliminate bulking. The average fineness moduli of the sand, trap-rock, and mix were 2.6, 7.8 and 5.6 respectively. The water-cement ratio was kept constant at 0.90.

Thermocouples were used for measuring the temperature of the concrete. Three of these were embedded in the central pier which was 8 ft. high, 50 ft. long and 25 ft. wide. The thermocouples were placed 4 ft. from the bottom on a line perpendicular to the center of the 50-ft. face, and at distances of 8.5 ft., 12.5 ft. and 16.5 ft. from that face.

Three thermocouples were also installed in each of the two shafts of the shore piers. These shafts were 16 ft. high, 14 ft. wide and 15 ft. long. The thermocouples were placed 3 ft. from

the bottom on a line perpendicular to the center of the 15 ft. side at distances of 1, 5 and 9 ft. from that side. All pourings were continuous at the rate of about 20 cu. yd. per hour.

Before any pouring was started, the forms were thoroughly warmed by salamanders. The sand, trap rock and water were heated by means of steam pipes. When each pour was finished, the entire section was covered with a tent-like arrangement of tarpaulins within which were placed salamanders heated with charcoal. Heating of the center pier was continued for 33 hours, of the downstream shaft of the shore pier for 54 hours, and of upstream shaft for 72 hours.

The records for the center pier show that the concrete was placed at 60 degrees F. when the surrounding air was at 40 degrees F. Between 8 and 38 hours after placing, the surrounding air averaged about 65 degrees F., and during this period the temperature of the concrete at all thermocouples rose 35 degrees to a maximum of 95 degrees F.

In the upstream shaft of the shore pier, the concrete was placed at a temperature of 45 degrees but fell to 40 degrees F., the temperature of the surrounding air, in 3 hours. Thereafter, the temperature of the air averaged about 65 degrees for 70 hours, at the end of which period the temperature 1 ft. from the face reached a maximum value of 92 degrees F. The temperature at 5 ft. from the surface continued to rise to a maximum value of 100 degrees F. at the end of 140 hours, at which time the air temperature was 30 degrees F.

In the downstream shaft of the shore pier, the concrete was placed at a temperature of 75 degrees but at the end of two hours fell to 69 degrees, when the air was at 38 degrees F. The air temperature gradually increased to 75 degrees F. at the end of 40 hours, and then fell to 66 degrees at the end of 54 hours. In the meantime, the concrete 1 ft. from the face rose to a temperature of 102 degrees F. at the end of 49 hours, and the temperature 5 ft. from the face increased to 105 degrees F. at the end of 54 hours.

PARDEE DAM

The Pardee dam, constructed in 1928, is a gravity type arch structure 350 ft. high above stream bed. The width at the base

of the dam is 244 ft., and the width at the top is 16 ft. The length of the dam is 1337 ft. and the volume of concrete is 615,000 cu. yd.

The concrete contained 4 sacks of Calaveras cement per cu. yd. and the aggregate in the mix, measured by volume, consisted of 30 per cent of 0 to ⅜-in. sand, 28 per cent of ⅜ to 1½-in. gravel, and 42 per cent of 1½ to 6-in. cobbles. Enough water was used to produce an average slump of 6 in.

In December 1928, when the dam had reached an elevation of 180 ft. above stream bed and 170 ft. below the top, five thermocouples were embedded in a line along the center of a block 75 ft. wide, located just south of the center line of the dam. The thermocouples were at 2 ft., 12 ft. and 55 ft. from the upstream and downstream faces, the latter point being at the center of the dam, as at this elevation the dam is 110 ft. thick. After placing the thermocouples, the concrete was poured in 5-ft. lifts at the rate of about one lift every 5 days.

The concrete was placed at a temperature of 71 degrees F. when the outside air was 39 degrees F. The temperature at the point 55 ft. from the two faces of the dam continued slowly to increase until it reached 102 degrees F., an increase of 31 degrees F., nearly seven months later when the air temperature was 77 degrees F. The average air temperature during this interval was 57 degrees F. The maximum rise in temperature 12 ft. from the face was 36 degrees F., occurring within 20 days, and the maximum rise 2 ft. from the face was 27 degrees, occurring within 12 days.

RESERVOIR WALL

In a discussion on high early strength concrete, H. V. McCall[19] states that in a series of tests upon a reservoir under construction during the winter where the concrete was placed at an initial temperature varying from 55 to 75 degrees F., the temperature developed within 3 days, in sections about 8 ft. thick, was approximately 60 degrees F. higher than when the concrete went into the forms.

PUTNAM COUNTY ARCH BRIDGE

Setting temperatures were determined during the construction of an arch bridge near Eatonton in Putnam County, Georgia[20].

This structure has an open spandrel arch with a span of 160 ft. and a rise of 46 ft.

The proportions of the concrete mix were 1:2:4 by volume, the fine aggregate under ¼ in. being a well-graded, clean quartz and the coarse aggregate, which varied from ¼ to 1½ in. in size, consisting of 50 per cent of quartz gravel and 50 per cent blast furnace slag. The time of mix was 2 to 5 minutes and the slump varied from 3½ to 4½ inches. The concrete was deposited in batches of about ⅓ cu. yd.

Temperatures were observed by means of electrical resistance thermometers embedded at different points within the mass. As typical of the temperature record, the concrete at the center of a quarter-point of an arch rib developed a maximum temperature of 131 degrees F. after 7 days, the initial temperature having been 77 degrees F. and the air temperature being 73 degrees F. at the time of occurrence of the maximum temperature in the concrete. The size of the arch rib at this point is not reported.

CHUTE A CARON DAM

Tests of the setting temperatures of the concrete entering certain auxiliary structures in connection with the Chute a Caron Dam in Quebec, Canada, are reported by I. E. Burks.[21]

The concrete in these structures had a water-cement ratio of 0.90 and the mix was approximately 1 part cement, to 2.3 parts of sand, to 4.8 parts of crushed stone by volume. The pit sand varied in fineness modulus from 1.9 to 3.6. The crushed stone was a feldspar obtained at the dam-site ranging in size from 2 to 6 in., and having a fineness modulus varying from 8.8 to 9.5. The concrete had no particles between ⅜ in. and 2 in. The compressive strength of the concrete was about 3,000 lb. per sq. in.

After a 2 to 3-minute mixing period, the concrete was hauled to the dam in 4-yd buckets, and placed with the aid of vibratory tampers. The time interval between mixing and placing varied from 15 to 50 min.

In winter, the concrete materials were heated so that the concrete left the mixer at from 70 to 95 degrees F. and in some instances lost as much as 8 degrees before it was placed in the forms. After placing, the concrete was kept above 60 degrees F.

for 72 hours by covering with tarpaulins and heating with steam coils.

The temperatures within the concrete were observed by placing thermometers in wells made of ½ in. pipe from 3 to 8 ft. long. Concrete placed at a temperature of 78 degrees F. in Penstock No. 1, which is a mass 8 ft. thick, rose to 112 degrees F. at 2 in. from the exposed face at the end of 36 hours and the temperature at the center of the mass rose to 132 degrees at the end of 66 hours. The temperature of the air under the tarpaulin is not stated, but the outside air averaged about minus 5 degrees F.

In an obelisk pier, concrete placed at a temperature of 90 degrees rose to 157 degrees F. at a depth of 30 in. within 12 hours after placing. The air temperature outside the tarpaulin, averaged about 22 degrees F. during this interval.

TESTS AT THE UNIVERSITY OF CALIFORNIA

In 1929, under the direction of R. E. Davis and G. E. Troxell, several series of tests which have not been previously reported were carried on at the University of California, to determine not only the maximum temperature developed in mass concrete, but also to determine the effect of the high setting temperatures upon the compressive strengths at various ages.

For the first series of tests there were employed three different mixes of gravel concrete, as shown in the following table. The consistency of all mixes was kept constant at a 4-in. slump.

Mix No.	Water Gal. per Sack	Water-Cement Ratio	Approx. Mix	Proportions by Volume			
				Santa Cruz Cement	Sand	¼ to ¾ in. Pebbles	¾ to 1¼ in. Pebbles
1	4½	0.60	1:3	1	1.20	0.90	0.92
2	6	0.80	1:5	1	2.11	1.54	1.57
3	8	1.07	1:7	1	2.82	2.07	2.08

The sand was a well graded material varying from 0 to ¼ in. and having a fineness modulus of 3.03. The ¼ to ¾-in. pebbles and the ¾ to 1¼-in. pebbles had fineness moduli of 6.33 and 7.81 respectively.

TABLE 3—TEMPERATURES OF CUBES AND STRENGTHS OF CYLINDERS (DAVIS AND TROXELL)

Mix No.	Approx. Mix	Proportions by Volume	Gal. Wtr. Per Sk. Cement	Cylinders	Storage	Room Temp. °F.	Cube Temp. °F.			Hours to Max. Temp.	Compressive Strength, lb. per sq. in.					
							Initial	Max.	Rise		1-Day	3-Day	7-Day	28-Day	90-Day	1-Year
1	1:3	1:1.20:1.82	4.5	Inner	Cube	77	71	154	83	33	3110	3130	3720		4120[2]	4030
1	1:3	1:1.20:1.82	4.5	Outer	Cube	77	71	153	82	33	3080	3270	3710		3960[2]	3950
1	1:3	1:1.20:1.82	4.5	Control	Lab. Air[1]						995	2530	3200		5010[2]	
1	1:3	1:1.20:1.82	4.5	Control	Fog at 70°						1375	3000	3870		5750[2]	5220
2	1:5	1:2.11:3.11	6	Inner	Cube	68	60	115	55	94			3340	3840	3795	
2	1:5	1:2.11:3.11	6	Outer	Cube	68	60	115	55	94			3260	3780	3770	
2	1:5	1:2.11:3.11	6	Control	Fog[3]								2120	3620	4170	
2	1:5	1:2.11:3.11	6	Control	Fog at 70°[4]								2610	4230	4570	
3	1:7	1:2.82:4.15	8	Inner	Cube	67	59	101	42	97				2890	3240	3280
3	1:7	1:2.82:4.15	8	Outer	Cube	67	59	101	42	97				2890	3160	3360
3	1:7	1:2.82:4.15	8	Control	Fog[3]									2620	3420	
3	1:7	1:2.82:4.15	8	Control	Fog at 70°[4]									3180	3850	3830
3	1:7	1:2.82:4.15	8	Control	Air at 70°[5]											4050

[1]Specimens in air at about 60 degrees F. stored in paraffined cardboard molds with slip covers.
[2]After 7 days storage as indicated, cylinders were removed, coated with asphalt and stored at 70 degrees F. and 70% relative humidity.
[3]Specimens stored in fog at 70 degrees in paraffined molds without covers, for 28 days. Cylinders then sealed with tin covers, coated with asphalt and stored at 70 degrees F. and 70% relative humidity.
[4]Molds removed after 1 day.
[5]Specimens in cardboard molds sealed with tin covers, coated with asphalt and stored at 70°F. and 70% relative humidity.

For the 1:3 mix (using 4½ gal. of water) three cubes, each 3 ft. on a side, were made. Each cube was cast in a wooden form made of 2-in. lumber, placed inside of a second form of 1-in. lumber with a 6-in. space between, this space being filled with dry sawdust. While pouring the cube, twelve 6 x 12-in. cylinders of the same mix, in paraffined cardboard cylinders with metal covers at each end, were embedded in the cube as close to the center as possible—four in an inner circle and eight in an outer circle. Several of the cylinders had iron-constantin thermocouples embedded in them so that the temperatures developed in the concrete might be observed. Other thermocouples were placed at various positions in the cubes in order to observe the temperature differentials between the various parts of the mass. After completion of the cubes, their tops were covered with waterproof building paper and with 6 in. of dry sawdust. Thereafter, frequent observations were made of the temperatures within the mass and of the air in the room where the cubes were stored.

At the same time that the cylinders were embedded in each cube, 10 similar specimens were made to serve as control cylinders. Of these control cylinders, 4 were left uncovered in a fog room at 70 degrees F., while the remaining 6 were stored under normal atmospheric conditions at about 60 degrees F.

Four similar cubes, except for a 12-in. insulation of sawdust instead of 6 in., were made of the 1:5 mix (using 6 gal. water) and 3 were made of the 1:7 mix (using 8 gal. water). Some of the control cylinders for these cubes were removed from their forms after 24 hours and stored in the fog room at 70 degrees F. The remaining control cylinders were sealed in molds with tin covers, the entire mold coated with an asphalt emulsion, and the specimens then placed in the fog room at 70 degrees F.

The cubes were broken into fragments at ages varying from 1 day to 1 year, and the embedded 6 x 12-in. cylinders were tested in compression together with the corresponding control cylinders for the cube.

The critical data regarding setting temperatures of the cubes and the strengths of the 6 x 12-in. cylinders are given in Table 3.

From a study of the data of Table 3, it will be observed that the compressive strength of rich concrete cured in mass at high tem-

peratures is high, even at the early age of 1 day. It will also be noted that the gain in strength thereafter is slow. These data show that the use of rich mixes of portland cement concrete to obtain high early strength is feasible if provision is made to prevent the dissipation of the heat of chemical reaction for the first day or so. However, there is no advantage in retaining the chemical heat of rich mixes if high strength is not desired before 3 days, since at that age, concrete water-cured at normal temperatures would have equal strength. The superiority of concrete water-cured at normal temperatures over concrete cured in mass at high temperatures is plainly shown by the results of the 28 and 90-day and 1-year tests.

While observing the temperatures developed in the concrete cubes, it was noted that the temperature drop between the inner cylinders and a point 1 in. from the face of the cubes amounted to as much as 13 degrees F. for mix No. 1, whereas it was only 2 degrees F. for the other mixes. Part of the difference is undoubtedly due to the use of only 6 in. instead of 12 in. of sawdust for the richer mix, but some of the difference is due to the higher temperature attained by Mix No. 1.

A second series of tests was conducted using the two richer mixes of the first series. Three groups of cylinders were made for each mix. The first group were sealed in cans and subjected to the same temperature variations as the concrete in the 3 ft. cubes of series 1. The specimens of the second group were stored without forms in the fog room at 70 degrees F., while those of the third group were sealed in cans and kept at 70 degrees F.

For Mix No. 1, the specimens cured at the high temperatures gained strength quite rapidly for 3 days. A slight further rise was shown at 7 days, but some retrogression in strength was shown between 7 and 28 days. No further retrogression occurred at 90 days. The fog cured specimens, and those sealed and cured at 70 degrees F., gained in strength consistently during the 90-day period, the fog cured specimens indicating about 15 per cent higher strength than those sealed at 70 degrees F. The strength of the latter groups surpassed that of the first group at the ages of 28 and 90 days.

For Mix No. 2, the specimens cured at the high temperatures gained in strength for 28 days, and then showed a slight retrogression at 90 days. The other two groups gained strength more slowly at first, but attained nearly the same strength as the first group at 28 days. At 90 days, the strength of the fog cured specimens exceeded that of the cylinders cured at the higher temperatures.

In order to determine the effect of the brand of cement upon the maximum setting temperature and to ascertain the effect of this temperature upon the compressive strength, Davis and Troxell conducted a third series of tests upon 9 different brands of portland cement.

The proportions by weight of the concrete mix for these tests were 1:1.50:1.85, with a water-cement ratio of 0.67 by volume and a slump of 3 to 3½ in. The sand was well graded from 0 to ⅜ in. and had a fineness modulus of 3.71. The coarse aggregate was a crushed stone graded from No. 14 mesh to ½ in. and had a fineness modulus of 5.82.

Four 18-in. cubes were poured at one time. These 4 cubes were placed in a single large enclosure about 6 ft. square and 3½ ft. deep so that each 18-in. cube could be insulated with 12 in. of dry sawdust on all sides, and on top and bottom. Thermocouples were embedded at the center of each cube to observe the setting temperatures. The air in the room where the tests were in progress was kept constant at 70 degrees F.

The results of these temperature observations are shown in Table 4.

The compressive strengths of the concretes made from these 9 cements under various curing conditions were obtained by filling 45 tin cans 3 in. in diameter and 6 in. high, with each of the 9 mixes. Tin covers were fitted to the cans after filling. Of these 45 specimens made with each cement, 15 were stored in a room at 70 degrees F. and 50 per cent relative humidity, 15 were stored in fog at 70 degrees F., and 15 were stored in a high temperature chest in which the temperature was varied according to a time-temperature curve in close conformity with the setting temperatures of the mass concrete made of the various mixes. The 9 brands of cement were separated into three groups

TABLE 4—SETTING TEMPERATURES OF CONCRETE MIXES USING PORTLAND CEMENTS (DAVIS AND TROXELL)

Group	Brand	Temperatures, °F.			Age at Max. Temp., Hrs.
		Initial	Max.	Rise	
A	1	68	125	57	33
	2	65	127	62	32
	3	69	121	52	28
	4	68	120	52	40
B	5	64	128	64	25
	6	69	130	61	28
	7	66.5	132	65.5	32
C	8	69	133	64	30
	9	68.5	139	70.5	29

TABLE 5—COMPARISON OF COMPRESSIVE STRENGTHS AT VARIOUS AGES OF NORMAL TEMPERATURE CURE WITH HIGH TEMPERATURE CURE SIMULATING MASS CONCRETE CONDITIONS (DAVIS AND TROXELL)

Group	Brand of Cement	Fog Room at 70° F.			High Temperature		
		1-Day	3-Day	7-Day	1-Day	3-Day	7-Day
A	1	545	1355	2400	1050	2060	3080
	2	580	1925	2930	1755	2340	2645
	3	1190	3220	3930	3065	4090	4670
	4	530	1910	2990	1700	2380	2610
B	5	1060	1960	2985	2890	3485	3755
	6	865	2135	3290	2375	2765	3380
	7	845	2455	3550	2395	3020	3170
C	8	1310	2890	4020	2715	4100	4220
	9	1230	2900	3940	2640	3425	3450

Groups A, B, and C cured at maximum temperatures of 163, 173 and 180 degrees F. respectively.
Compressive strengths in lb. per sq. in.

—A, B and C—as shown in Table 4, and all specimens of each group were subjected to the same heat treatment.

Due to the fact that there was not a large difference in average maximum temperatures for the three groups, it was decided arbitrarily to raise the average maximum temperature of Group C and then to raise the maximum temperatures of the other groups

proportionately, but still to retain the same age in attaining the maximum temperature and the same shape of the time-temperature curves. This change in the maximum temperature was made to see if the differences, if any, in the cements could be better shown. A maximum temperature of 180 degrees F. was assumed for Group C. This established the maximum temperatures for Groups A and B at 163 and 173 degrees F. respectively.

Five cylinders of each mix were tested at the ages of 1, 2 and 7 days; the results of the tests being shown in Table 5.

From a study of the data, it will be observed that for all tests at the ages of 1 and 3 days, the cylinders cured at the high temperatures exhibited higher strengths than those cured at 70 degrees F. At 7 days, the strengths of the high-temperature cured cylinders exceeded the strengths of the cylinders stored at 70 degrees F. for 5 cements out of the 9 tested.

PORTLAND CEMENT ASSOCIATION TESTS

The results of a series of tests made in the laboratory of the Portland Cement Association (submitted by McMillan by letter), which have a bearing on the effect of temperature in the concrete mass on the compressive strength, are shown in Table 6. The specimens were all made from materials at room temperature, then placed in steel forms with cast-iron base and cover plates and submerged in water at various temperatures for the entire period of the test, the forms being removed after 24 to 48 hours. This storage permitted the specimens to acquire the temperature of the surrounding water which could be easily measured.

From a study of the data in Table 6, it is seen that at all ages tested, there is a noticeable increase in strength for all temperatures up to 212 degrees F., with the one exception of the 28-day strength for the highest temperature. For the 28-day tests, the concrete cured at 125 degrees F. showed a slightly greater strength than that cured at 212 degrees F.

CEMENT PASTE TESTS MADE AT U. S. BUREAU OF STANDARDS

Tests were made by P. H. Bates[22] at the U. S. Bureau of Standards upon 32 brands of portland cement to determine the maximum temperatures attained during the setting process and to ascertain the rate of temperature rise. The cements tested were

TABLE 6—EFFECT OF TEMPERATURE OF STORAGE ON THE COMPRESSIVE STRENGTH OF CONCRETE (McMILLAN)

Temperature of Storage, °F.	Compressive Strength, lb. per sq. in.			
	2-Day Storage	3-Day Storage	7-Day Storage	28-Day Storage
33	150	250	830	2480
42	280	410	1210	2690
65	740	1140	1950	3430
125	1680	2020	2710	3690
212	2000	2360	2930	3480

those selected for other investigations and reported by Committee C-1 of the American Society for Testing Materials (Proc. A. S. T. M., 1928, Part I, p. 233).

In conducting these tests 200 grams of cement were mixed with 67.5 grams of water by a high speed motor-driven stirrer for one minute, in a tinned can $2\frac{3}{8}$-in. in diameter and 3 in. high. This was then placed in a thermos jar, closed and placed in a cabinet maintained at 70 degrees F. for 24 hours. The temperature was read by multiple junction base metal thermocouples and recorded by a sensitive potentiometer.

The time-temperature curves plotted from the observed data show an appreciable difference between the rates of temperature rise and the maximum temperatures attained, the latter values being shown in Table 7.

CEMENT PASTE TESTS BY MERRIMAN

Tht same cements used at the Bureau of Standards were also tested by Thaddeus Merriman, Chief Engineer of the New York Board of Water Supply[22]. The procedure followed was to mix 450 grams of cement with 112 ml. of water in a tinned can, 3 in. in diameter and 3 in. high. This was placed inside of a specially constructed calorimeter and the temperature of the mixture determined by the use of thermometers read at 15 minute intervals. The heat losses of the calorimeter were also determined at different temperatures and the temperatures read on the ther-

TABLE 7—TEMPERATURE RISE OF NEAT CEMENT PASTES

Cement No.	Results of Tests by Bates		Results of Tests by Merriman	
	Max. Temp. Rise Deg. F.	Hours to Develop Max. Temp.	Rise at 16 Hrs. Deg. F.	Rise at 24 Hrs. Deg. F.
1	77	11	196	198
2	31	20	124	140
3	47	10	158	158
4	33	15	138	149
5	63	13	170	171
6	45	13	176	184
7	81	12	171	174
8	39	14	172	174
9	30	17	159	159
10	47	12	138	147
11	35	15	172	180
12	41	15	135	151
13	52	11	180	187
14	41	13	156	176
15	50	17	169	171
16	32	13	171	179
17	41	16	182	185
18	34	12	160	172
19	30	13	142	154
20	18	14	100	118
21	68	13	174	177
22	70	13	205	208
23	45	15	159	162
24	88	10	153	153
25	52	11	141	149
26	36	15	147	166
27	48	12	155	156
28	37	11	121	127
29	37	16	162	171
30	45	14	159	159
31	45	13	142	151
32	32	16	129	138

mometer corrected according to the previously determined heat losses. The results are presented in Table 7, along with those obtained at the Bureau of Standards.

There is no assurance that the maximum temperatures developed in large masses of concrete made from the 32 cements of Table 7 would have the same relationships as for the neat cements in small amounts. Some cements which indicated a slow liberation of heat and a relatively low maximum temperature in the tests by Bates and Merriman, might have shown

much higher temperatures relative to those attained by the other cements, if used in large masses of concrete; and the reverse might be true for those cement pastes which exhibited high temperatures.

PROPOSED EXPERIMENTAL PROGRAM

From a review of what has gone before, it appears that very little is known concerning the properties of concrete in large masses. To determine these properties and the influence of various factors upon them obviously is not a simple matter, first by reason of the large number of variables involved, second because we are dealing with large volumes, and third because of the necessity of long-time studies.

It is believed that a comprehensive series of laboratory tests should be made to determine the effect upon: (a) rise in temperature, (b) compressive strength at various ages, (c) modulus of elasticity at various ages, (d) flow at various ages, (e) residual strains, (f) residual stresses, (g) permeability, (h) durability, and (i) volume changes as affected by: (1) chemical composition of cement, (2) fineness of cement, (3) admixtures, (4) richness of mix, (5) water-cement ratio, (6) initial temperature, (7) character of aggregate, and (8) size of mass.

Supplementing the laboratory tests there should be field investigations which may be properly divided into two parts: first, those which have to do with structures already built and in service; and, second, those concerning new work where conditions as regards materials and methods of construction may be closely controlled. It is clear that one of the very important functions of the committee is to promote the cooperation of those in charge of construction enterprises from which pertinent information may be derived. Acting as a guide, the committee should be in a position to do much in promoting field research and preventing duplication of effort.

The principal masses with which the committee is interested are dams, but heavy foundations, bridge piers, massive arches, large anchorages, and thick tunnel linings ought not to be overlooked, for, because of the use of rich mixes, the temperatures of these latter structures may considerably exceed those normally reached in dams.

It is evident from the summary presented earlier in this report that, aside from records of temperature, there has been published comparatively little quantitative data concerning the behavior of mass concrete structures. However, during the past few years attempts have been made to measure dimensional changes as well as temperatures in a number of dams in this country, using the electric telemeter for determining interior changes and the strain gage for observing surfaces changes. While it appears that many of the unpublished data thus obtained are not reliable, owing to the inexperience of observers or to the inaccuracies of measuring instruments, information from these sources should be brought together and the results summarized.

Of considerable value also would be the principal facts concerning the physical condition of dams and other mass structures which have been in service for some time (particularly the location and width of cracks) together with details regarding the concrete materials and the manner of mixing, placing and curing.

The question of whether or not there is a gradual shrinkage, over a long period of time, of dams located in arid regions due to a gradual dissipation of uncombined water is one of considerable importance. Strain-gage measurements should be begun on a considerable number of these structures when the facts concerning the quality of the concrete are known, and these measurements should be extended over several years.

Attention should be given to developing a test program calculated to produce reliable and fairly complete information concerning the behavior of mass concrete in actual structures, particularly dams, under the wide variety of conditions met in practice, beginning with the construction period and extending over at least ten years. In order best to coordinate this field work it is considered imperative that the program as a whole shall be carried out under the general direction of a single organization, with the cooperation of those in charge of the individual enterprises. Insofar as possible, the field tests should supplement the laboratory program, being calculated to furnish quantitative data regarding the effect of variations in materials and processes, and in size and shape of mass.

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Readers are referred to the JOURNAL *for May, 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931.*

Introducing Two Papers on Slump Tests and Flow Table Determinations

Here we discover two pairs of experienced investigators, working independently, studying slump tests and flow table determinations as measures of that yet-to-be-satisfactorily-measured characteristic of concrete mixtures—workability. They have both established the important fact that relation of flow to slump is not constant—that the slump is relatively larger for the richer mixtures. Yet they arrive at opposite conclusions. Smith and Benham are for the flow table; Lyse and Johnson are for the slump test. Are there not other seekers after this important yard-stick of that characteristic, workability (which is now written into specifications, yet lacks definite clues to its identity) who, in the combined data reported in the two papers which follow, can discover a common ground —a reconciliation of interpretation, which to one authority is one thing and to the other something different? As a welcoming gesture toward such a reconciliation, the two papers, submitted to the Institute within five days of each other, are published together, and the discussion (May 1931 Journal) *will be combined. One member of the Publications Committee, while approving each paper in turn as a worthy contribution deserving publication, considers it an unavoidable imposition on the membership to publish two, where, under happier coordination, one might have sufficed. If in the ensuing discussion there should be even an approach to that coordination, the "imposition" may be justified.*—Editor.

A Study of the Flow-table and the Slump Test

BY GEORGE A. SMITH* AND SANFORD W. BENHAM†

The slump test, long used as a suitable and convenient means for measuring the consistency or wetness of concrete is recognized, nevertheless, as only a rough test for determining that condition or property. In the laboratory where more accurate control of the consistency is desirable the flow-table is frequently used and is generally looked upon as being the more satisfactory. There is always, however, considerable difficulty in attempting to compare flow-table data obtained by different experimenters because there is no standard method of operating the apparatus. The size, weight, height of drop, and speed of operation are not standardized. In order that wetness of concrete may be visualized from flow-table data it is imperative that the data be obtained under fixed conditions, as is the case with slump test data. Before such a condition can be brought about, information as to the functioning of the flow-table under different conditions is necessary.

In view of renewed interest in this subject the authors feel that the data here presented not only are opportune but point out characteristic tendencies which should be brought to the attention of others. It is believed that the results of this series of tests may be helpful torward establishing a standard method for performing the flow-table test.

PURPOSE AND SCOPE

It has been pointed out by several investigators that for a given condition of wetness as determined by the flow-table those mixes which contain the larger amounts of cement, and which are the more workable, quite consistently give the greater slumps. This condition has been observed by the authors for a long time

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and it has been felt that where the laboratory is endeavoring to simulate, in many ways, actual field conditions, either the same method of measuring the consistency should be used or a method which, though on a different basis, will give comparable results.

The tests with which this paper deals were made to determine the necessary flows to be used to give definite slumps so that more careful measurement of the wetness of the concretes could be made using the flow-table instead of the slump test. Though 15 ⅛-in. drops has been our usual measure for the work done in making the flow-table test, the program was extended to include both five and ten ½-in. drops because it was thought that the more vigorous action of the greater drop might give results paralleling the slump test more closely than did results with the ⅛-in. drops, and also because the ½-in. drop is used in a number of other laboratories.

The data were obtained on five mixes, from a 1:3:6 to a 1:1½:3, gaged to three slumps within the range 2 in. to 8 in. Consistency determinations were made using the slump test and the three different methods of making the flow-table test. The work was carried through all conditions both with and without Celite as an admixture. Incidental to the studies of consistency, the concrete was cast into cylinder specimens and tested in compression at 28 days.

In all, the study involved the making of 300 slump tests and 450 determinations of the flow and the testing of 300 compression specimens.

MATERIALS AND TEST METHODS

The cement used in this study was part of a 50-barrel lot of portland cement purchased from a local dealer. On delivery the cement was emptied on a concrete pavement, turned over twice and then stored in steel drums until used. Physical tests indicated that the cement met A. S. T. M. standard specifications for portland cement.

Air-dry, washed bank sand from Spring Lake, N. J., was used as fine aggregate. The sand was graded 0 to No. 4, had a fineness modulus of 2.90, and weighed 103.8 pounds per cubic foot. The sand as received was separated on the No. 8 and the No. 16 sieves.

Five sieve analysis were made on each fraction and the three sizes combined to give the desired fineness modulus when preparing individual batches.

The coarse aggregate consisted of air-dry, washed gravel from the same source as the sand. This was separated into four sizes and later recombined. The combined gravel was graded No. 4 to 1 in. (square openings), had a fineness modulus of 7.04, and weighed 105.2 pounds per cubic foot. Neither the fine nor coarse aggregate contained appreciable organic impurities and in an air-dry state showed practically zero absorption.

In all mixes containing an admixture commercial Celite for concrete was used.

PROPORTIONS

All mixes were proportioned on a dry rodded volume basis for the fine and coarse aggregate, using 94 lb. per cu. ft. as the unit weight of the cement. Throughout the study the ratio of sand to gravel was one to two by volume and the quantity of aggregate per batch was constant, viz., 0.10 cu. ft. (4.709 kg.) of sand and 0.20 cu. ft. (9.539 kg.) of gravel. To effect variations in the richness of the mix the quantity of cement alone was varied to obtain the following mixes:

Mix (by Volume)	Per Cent Cement	Cement Used (Kgs.)	Celite Content Lbs. Per Sack
1:1½:3	133	2.843	2.25
1:1⅔:3⅓	120	2.558	2.50
1:2:4	100	2.132	3.00
1:2½:5	80	1.706	3.75
1:3:6	67	1.421	4.50

Based on the cement content of the 1:2:4 mix these mixes represent cement contents varying from 67 to 133 per cent as indicated in the table above.

When Celite was used as an admixture the quantity used per batch was constant (68 grams). The resulting Celite content, in pounds per sack of cement, and the quantity of cement used per batch are also shown in the table above.

These quantities resulted in a quantity of concrete sufficient to fill the slump cone with a small excess.

MAKING TESTS

Without going into specific detail as to making the tests, suffice it to say that batches for one complete run of 30 mixes (a days' work) were prepared and standard practice followed in performing the remainder of the work. With regard to making the flow and slump tests some explanation, however, is necessary. Each of the five mixes was gaged to give what would appear to be 2-, 6- and 8-in. slumps. The first run was made determining the flow with 15 ⅛-in. drops followed immediately by a slump test. The second run, on the day following, was made duplicating all conditions except for the manner of making the flow test, when five and ten ½-in. drops were used instead of 15 ⅛-in. drops. In obtaining the flow, using ½-in. drops, the specimen was first jolted five times, measured and then given five additional jolts and the diameter (average of four) again determined. This procedure resulted in ten slump tests and five flow-tests for each of the three different methods of operating the flow-table. The work involved the making and testing of ten separate batches of concrete for each condition as to mix and wetness. The flow is expressed as the percentage increase in diameter of the specimen.

After the consistency test was completed the concrete was cast in a 6 by 12-in. cylinder mold, capped when one day old and cured in a fog room at about 70° F. for 28 days at which time it was tested.

It is felt that because of the various types of flow-tables used in different laboratories a partial description of the table used in these tests is warranted. The apparatus was purchased from the H. C. Cragg Mfg. Co., Washington, D. C., and is similar to that designed by the Bureau of Public Roads and to the table used at the Bureau of Standards. Specifically the top weighs 122.5 pounds, has a diameter of 29.5 in., and a thickness of 0.5 in. It is made of bronze reinforced with eight ribs 0.25 in. thick, tapering from ⅜-in. near the circumference to a depth of 2.5 in. at the hub. The drop was changed from ⅛ in. to ½ in. by removing a washer

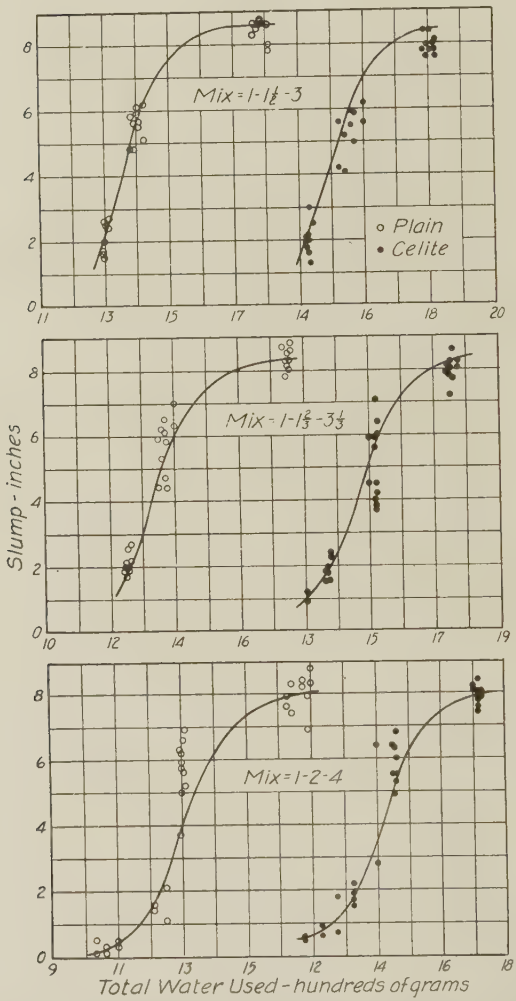


FIG. 1—RELATION BETWEEN SLUMP AND TOTAL WATER USED FOR VARIOUS MIXES

$\frac{3}{8}$ in. thick from the supporting shaft of the flow-table as used for the $\frac{1}{8}$-in. drop.

RESULTS OF TESTS

Rather than encumber this paper with tables of data, results of individual tests are plotted to show the grouping and variations in the data as well as the trends of the results. In all curves presented the open characters represent values for mixes without admixture and the solid characters those for concretes containing Celite. Figs. 1 and 2 show the slumps plotted against the actual weight of water used. Figs. 3 and 4 present similar data obtained for the three methods of using the flow-table.

The three methods of using the flow-table are again shown in Fig. 5 to indicate the change in water content for a given flow as the quantity of cement varies. Fig. 6 shows the change in flow for 15, $\frac{1}{8}$-in. drops and for the various mixes with change in water-cement ratio. All points indicated in Figs. 5 and 6 have been taken directly from the curves indicated in Figs. 3 and 4.

Fig. 7 shows the relation between the different methods of operating the flow-table based on data from all mixes.

Fig. 8 has been prepared to show the relation that existed between the slumps and the flows obtained when 15, $\frac{1}{8}$-in. drops were used.

Results obtained from the compression tests are shown in Fig. 9 where the average strengths from ten tests for each condition are plotted against the average water-cement ratio.

In Figs. 1 and 2 it may seem that part of the data in the region of intermediate slumps has been largely disregarded when drawing the curves. Such, however, is not the case. When experimental data are obtained using systematic variations in conditions the resulting trends obtained are usually systematic in their arrangement. It would, therefore, be natural to suppose that the relations of slump to flow would show the same or parallel trends when varying the mix. See Fig. 8.

In order to obtain the systematic arrangement of the curves shown in this figure it was necessary to make some adjustment of the water-slump curves. Since the trends of the flow data would permit little or no adjustment most of the adjusting was confined to the curves of water versus slump. Therefore, the

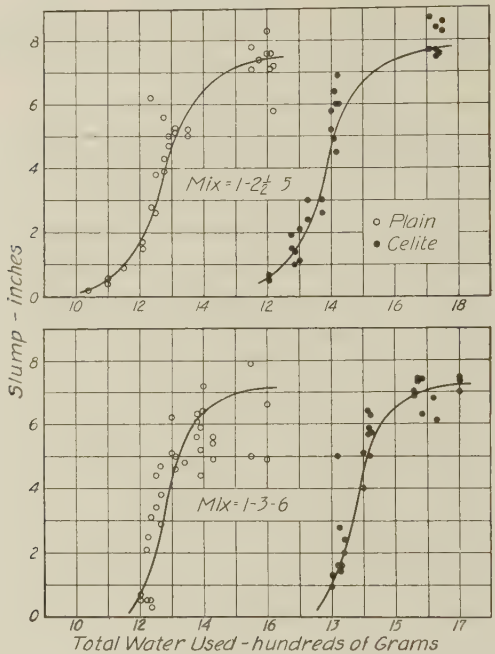


FIG. 2—RELATION BETWEEN SLUMP AND TOTAL WATER USED FOR VARIOUS MIXES

position and shape of a single water-slump curve, shown for any given mix, are governed not only by the data obtained for that mix but are influenced by the data obtained for all other mixes.

DISCUSSION OF RESULTS

The results obtained from the slump test (Figs. 1 and 2), though self-explanatory in many ways, call for some comment. It will be noted that the general shape of the curves is approximately that of a reversed curve with the point of inflection in the region of a 4-to 5-in. slump and limiting tangents at zero, probably, and in the region of 8 in. The curves are about the same shape as those shown by McMillan and Johnson[1] for similar mixes. For mortars and highly sanded mixes it is not improbable that the 8-in. upper limit may be increased as has been shown.[1]

[1] "Further studies of the Water-Cement Ratio Strength Relation in Concrete"—Report of the Director of Research, Portland Cement Association, November 1928, p. 16.

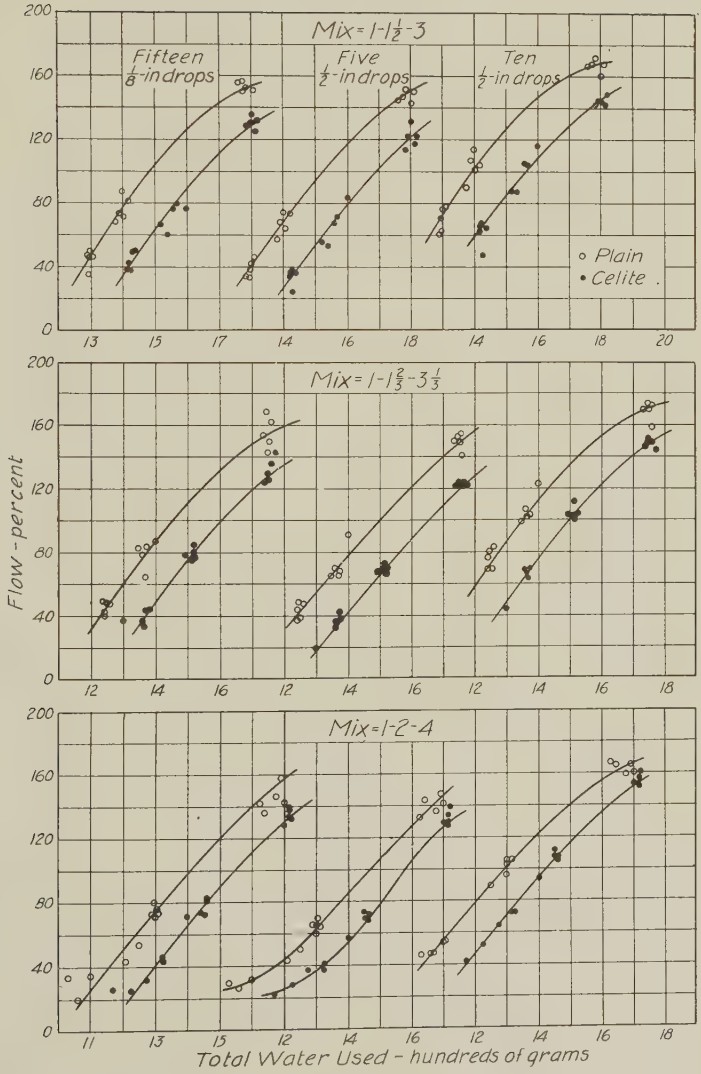


FIG. 3—RELATION BETWEEN FLOW AND TOTAL WATER USED FOR THREE METHODS OF OPERATING THE FLOW-TABLE

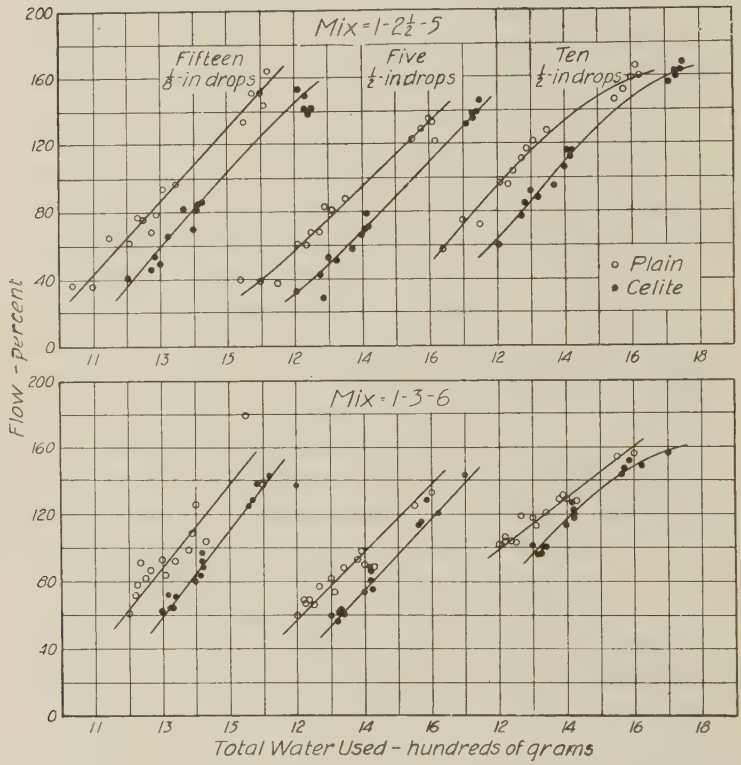


FIG. 4—RELATION BETWEEN FLOW AND TOTAL WATER USED FOR THREE METHODS OF OPERATING THE FLOW-TABLE

At both the low and high limits the test is less sensitive to change in water content than it is in the central range, if the condition indicated by the curves is taken as a criterion of the sensitivity of the test. In the range of intermediate water contents the individual data indicate that the test appears to function in a very uncertain manner.

During the making of the tests four characteristic conditions as to general shape of the resulting slump specimen were observed.

1. Specimen slumped only small amount and retained somewhat the same shape as the molded sample.

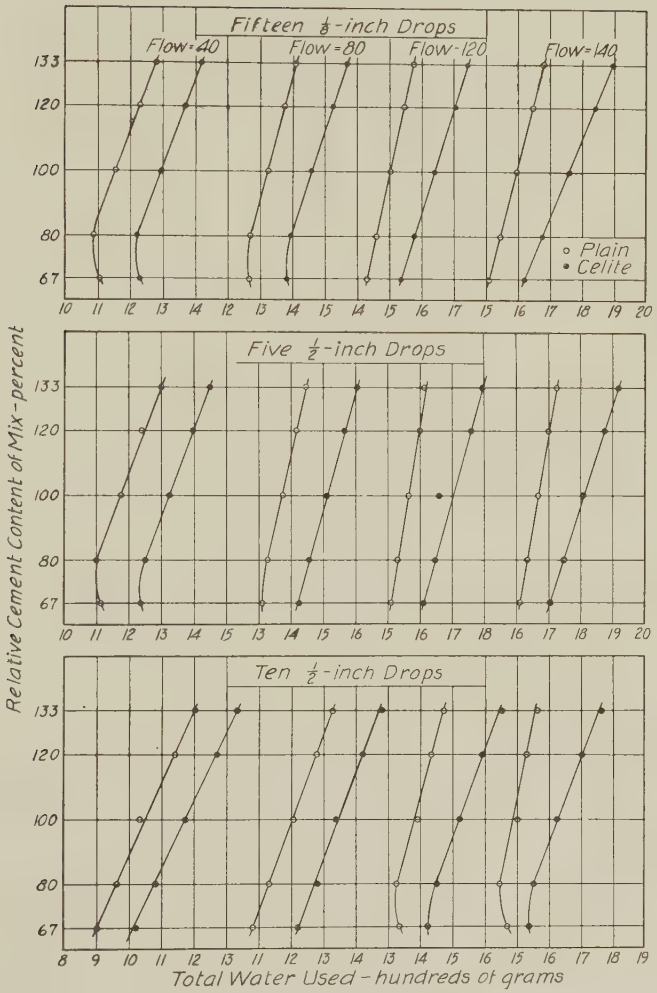


FIG. 5—RELATION BETWEEN THE RELATIVE CEMENT CONTENT OF MIX AND THE QUANTITY OF WATER REQUIRED FOR GIVEN FLOWS. CEMENT CONTENT OF MIXES IS BASED ON THAT OF A 1-2-4 MIX AS 100 PER CENT

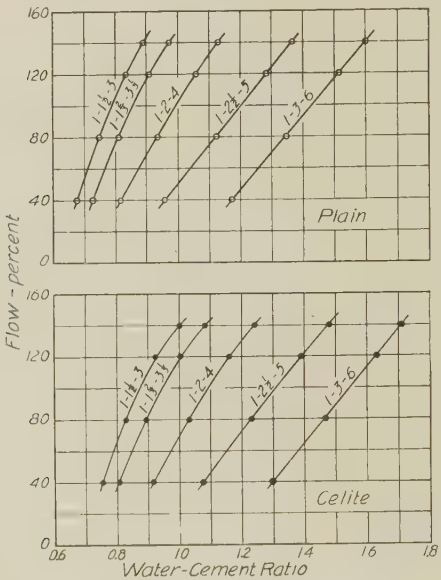


FIG. 6—RELATION BETWEEN FLOW AND WATER-CEMENT RATIO USING FIFTEEN ⅛-INCH DROPS OF THE FLOW-TABLE

2. Specimen slumped several inches but held together, bulging out uniformly as the slump took place.

3. A portion of the specimen stood up on one side and the remainder fell away. Not a usual condition and always checked by a second trial.

4. Top slushed down leaving a mound, generally of compacted aggregates, in the center.

In the region of 1- to 2-in. slumps, where the specimen conformed to Condition No. 1 above, the variation in slump for a given water content was relatively less than in other regions and in this region the slump test would appear to give some consistent indication of wetness.

In the upper region, 7- or 8-in. slumps, after a definite quantity of water had been used no change in the slump was produced by

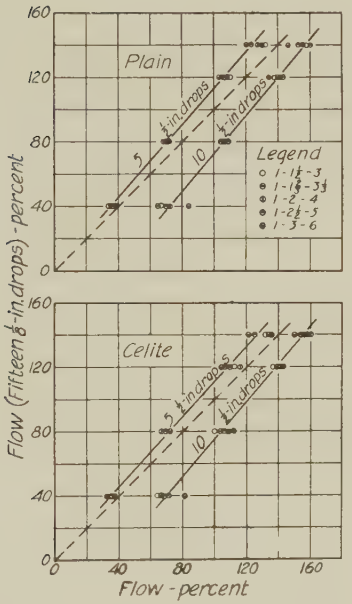


FIG. 7—COMPARISON OF DIFFERENT METHODS OF OPERATING THE FLOW-TABLE

increasing the water. In this case the test was characteristically that indicated by Condition No. 4. Due to segregation and compacting of the coarse aggregate a cone of approximately constant height was left as the upper part of the sample flowed down on all sides.

In the intermediate region of wetness (slump approximately 4-to 6-in.) the slump test was at its worst as is indicated by the wide range of slumps that were obtained with a constant quantity of water. Though the results generally showed somewhat better agreement in the richer and more workable mixes it was only in the richest mix considered that reasonable checks were obtained in this region. When trying for 6-in. slumps it was observed that the specimens assumed different shapes as represented by Conditions Nos. 2 and 3 or 4. That is, there appeared to be no con-

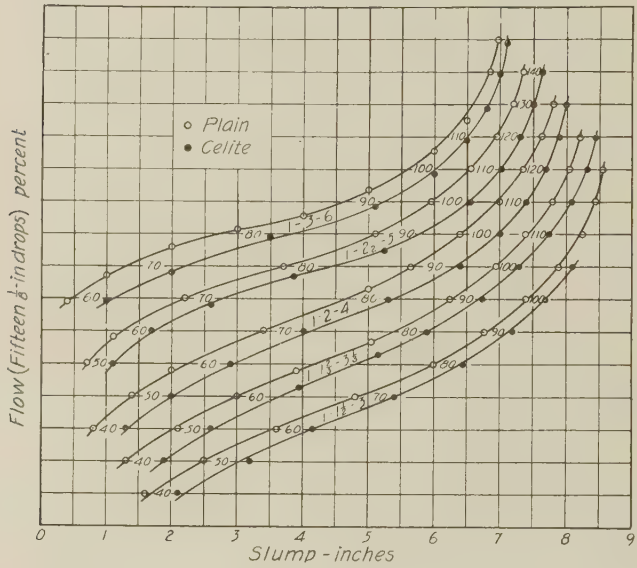


FIG. 8—RELATION BETWEEN FLOW AND SLUMP

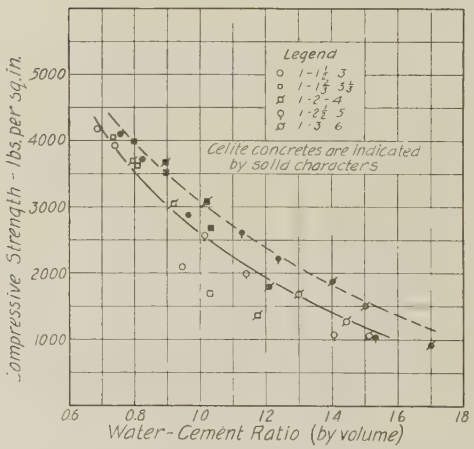


FIG. 9—RELATION BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH AT 28 DAYS

sistent performance as was generally noted in the 2-in. or the 8-in. slumps. If Condition No. 2 obtained the slump would be small. If No. 4 obtained the slump would be large. If No. 3 obtained the slump might be any amount in between the slumps given by Conditions Nos. 2 and 4. The fact that each curve shows a point of inflection would appear to indicate that in this region the test was ceasing to function in one manner and beginning to function in another. This appears to be consistent since for moderate slumps, 4 in. or less, the test is usually a true slump or a settling of the mass as a whole if we omit the lean, harsh mixes which in this discussion have been given less weight than have the plastic workable mixes. Beyond the true slump condition the wetness is such that the ability of the paste to hold the mass together is overcome and the mix acts more as a semi-fluid mass.

Individual results from the flow tests (Figs. 3 and 4) show consistent conformity to average curves, unusually so when compared with the results obtained with the slump cone. An exception to this is shown in Fig. 4 for the 1:3:6 mix. Here, as in the slump tests, it is believed that lack of quantity and quality of the cement paste resulted in extreme harshness and segregation which are always undesirable and certain to give erratic results. However, even in the lean mixes, the flow-table data are in better agreement than are the slump data.

Within the range that may be classed as plastic concrete the flow is practically a straight line function of the quantity of water used. This holds quite generally regardless of the mix and regardless of which method is used in making the flow test. In this connection it should be kept in mind that the volume of aggregate was constant and that different conditions of richness were obtained by varying only the quantity of cement. In other words the volume of concrete resulting from each batch was practically constant. In the upper reaches of the curves there appears to be more of a tendency for the curves to flatten indicating somewhat less sensitivity to increased water. This is particularly true in the richer mixes and in the case of the ten, ½-in. drops.

Relative to the functioning of the flow-table when using the different operations there appears to be but one outstanding

difference. This difference is only a question of the magnitude of the flow (see Fig. 7) and exists between the five, and the ten, ½-in. drops. It would be expected that the greater number of drops would increase the diameter of the specimen but that the increase in flow should be constant regardless of wetness, mix, or the presence of an admixture, does not at first appear to be logical. The parallelisms existing in Figs. 3 and 4 are brought out in Fig. 7 where the flows for the five, and ten, ½-in. drops are plotted against the flows for the 15, ⅛-in. drops. The fact that the curve of flows for the ten drops is parallel to that for the five drops indicates that when the first five drops have been administered the characteristics of a mix as to wetness or differences in consistency have been brought out and that the additional drops have no part in differentiating between conditions of wetness.

It was pointed out that it was thought the more violent action of the ½-in. drops might bring out differences similar to those that had been observed between the flow obtained with ⅛-in. drops and the slump test. Quite the opposite is to be noted. The fact that the slope of the curves for the ½-in. drops is steeper than the 45 degree line which represents the flow for 15, ⅛-in. drops (see Fig. 7) indicates that the ½-in. drop is not so sensitive as the ⅛-in. drop, since slightly greater changes in flow were obtained for the latter for a given change in consistency than were obtained for the ½-in. drops. Because the several points plotted, representing the five mixes, arrange themselves in no definite order it would appear that rich mixes show no action on the flow-table different from lean mixes. It will be noted that the curves in the lower portion of Fig. 7 coincide with those at the top of the figure indicating that the above discussion applies also to concrete in which Celite is used as an admixture. The conclusions with reference to the functioning of the flow-table are strengthened by the fact that in both group of tests, with and without an admixture, the same condition obtains.

The distinct parallelism that exists between the different operations of the flow-table is to be seen from a comparison of the curves in Fig. 5 in which richness or relative quantity of cement used is plotted against the quantity of water required to give a certain flow. Here the quantity of water varied uniformly with the richness of the mix irrespective of the method used. This

straight line relation between richness of mix and gaging water for a given consistency parallels other tests of similar nature by Prof. D. A. Abrams.[2]

An examination of Fig. 8 will indicate that previous observations pertaining to the relation of slump to flow have been confirmed where the richness of the mix is increased and when the admixture is used. For a given flow the slump showed a distinct tendency to increase with increased quantity of cement and with the use of Celite. Within the range of plastic mixes this increase in slump is closely proportional to the cement content.

Results of the tests do not take into account such conditions as disruption of the sample and segregation of the ingredients. In the case of the lean mixes the lack of paste resulted in considerable crumbling of the sample regardless of the method of operating the flow-table. This, however, was more pronounced for the ½-in. drop than for the less violent ⅛-in. drop. Even in the case of the stiff rich mixes the severity of the larger drop was evidenced by a more pronounced breaking up of the sample.

Although no study was made to determine the time required for performing the flow test by any of the three different methods it seemed to require no more time to make the 15, ⅛-in. drops than was required for the five, ½-in. drops. This was due to the necessity for stopping rotation of the hand wheel just as each half-inch drop occurred, to prevent the vertical shaft of the table from striking on the cam. Even if there were a saving in time of operation the authors feel inclined to recommend the use of the 15, ⅛-in. drops because of its slightly greater sensitivity to change in wetness, because the action is less violent, and because of the smoother and apparently better functioning generally.

Though the strength study made was purely incidental to the consideration of the slump-flow relation, the data are of some interest because of the tendencies and conditions brought out. In Fig. 9 the strength obtained for the different mixes (average of ten tests each) are distinguished by different notation. The two higher strengths for each mix were obtained for concretes gaged to slumps of approximately 2 in. and 6 in. These strengths

[2] "Tests of Powdered Admixtures in Concrete"—Proceedings Am. Society Testing Materials Vol. XXIX, Part II, p. 643, 1929.

alone have been used in locating the curves both of which are of the form $S = \frac{A}{B^x}$.

It is to be noted that, for the wide range of mixes considered, when the concrete was gaged to reasonable wetness or to give moderate slumps the resulting strengths conformed quite closely to a water-cement ratio-strength relation. When the slump was increased to about 8 in., requiring a larger amount of mixing water, the strengths did not conform to this relation and indicate that, for a given mix, the strength is approximately inversely proportional to quantity of water used. This relation appears to vary with the richness of the mix and to be more critical the richer the mix. This brings out the marked advantage to be obtained by limiting the wetness to moderate slumps in practice when high strengths are required. Increasing the wetness to the extent indicated in the case of the three richest mixes caused a 50 per cent reduction in the strength based on the potential strength at 28 days when moderate slumps were used. When reasonable consistencies are used the resulting strength may be indicated by the water-cement ratio-strength relation over a wide range of mixes.

With reference to the concretes in which Celite was used, it will be noted that the strength for a given water-cement ratio was higher when the admixture was used. When gaging the concretes to the same consistency it was necessary to add a certain amount of water to compensate for the absorption of the admixture. When correction is made for this it will be noted that the strength of the Celite concrete is equal to that of the plain concrete. It is to be noted that similar indications with regard to changes in water-strength relation with change in mix were obtained for the concretes containing Celite as were secured for the plain mixes.

CONCLUSIONS

Strength data obtained incidental to this study indicate that within the region of reasonable wetness, about 2- to 6-in. slump, the strengths conform quite closely to a water-cement ratio-

strength relation of the form $S = \frac{A}{B^{x}}$. Beyond the 6-in. slump, however, a separate water-cement ratio-strength relation appears to exist which varies with the mix and which is more sensitive to change in water in the richer mixes.

Though the concretes containing Celite produced higher strengths for a given water-cement ratio, when brought to the same slump by increasing the water somewhat to compensate for the absorption of the admixture, the strengths were equal to those obtained for plain concrete.

Although the slump test is without question more convenient for use in the field than is the flow-table, it is uncertain in its functioning particularly within the range where a sensitive and accurate determination of the wetness is desirable. This appears to be due to the slump's ceasing to function in one manner and beginning to function in another as evidenced by the shape or form of the specimen after slumping.

The results indicate that the flow test is more sensitive than the slump test in differentiating between changes in the wetness of concrete with changes in the quantity of water used. The change in flow within reasonable limits is approximately proportional to change in water regardless of the mix. For a given wetness or flow the quantity of water required is proportional to the quantity of cement used except possibly in the case of very lean mixes.

Using drops of different magnitude and varying the number of drops indicated that the change in flow with change in water was approximately the same for all methods used. The 15, ⅛-in. drops was somewhat more sensitive than the five or the ten, ½-in. drops, giving slightly greater increments of flow for a given change in water content.

For a given wetness or consistency as measured by the flow the slump changed considerably and quite consistently with change in richness of mix, increasing as the quantity of cement increased and when Celite was used to improve the workability.

The data on the whole indicate that, where a consistent indication of the consistency or wetness of concrete is desired, the flow test is preferable to the slump test because of its more nearly

uniform functioning. Because of easier action and greater sensitiveness to change in wetness the 15, ⅛-in. drops are to be preferred to either the five, or the ten, ½-in. drops.

A Study of Slump and Flow of Concrete

BY INGE LYSE* AND W. R. JOHNSON†

INTRODUCTION

The first efforts to control the quantity of mixing water in concrete were directed toward the development of some method of measuring the consistency. The slump test was the first method that met with any general application. This was followed by the development of the flow-table. Both these tests, which have been standardized by the American Society for Testing Materials, have been quite widely used for the control of consistency in laboratory investigations and to some extent in the control of consistency in the field. The slump test has achieved a much wider use in the field than has the flow-table, largely because of the greater simplicity in the apparatus required.

One of the earliest uses of these methods for measuring consistency was in the study of integral admixtures. Neither method, however, met with general favor in this field nor have they been widely accepted as suitable measures of the elusive property called workability. The deficiencies of these methods have been further emphasized by the recent development of the water-cement ratio specification for concrete in which the requirement of "plastic and workable" mixes is substituted for a direct specification of both grading and proportions of aggregates.

In spite of the recognized deficiencies of these methods, there is still much to commend them, and pending the development of a more applicable form of measurement it seemed desirable to investigate somewhat more thoroughly than has been done heretofore, the possibilities of these methods and to obtain a direct comparison between the two. The tests reported in this paper were undertaken with this in view, as a part of a general investiga-

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tion of admixtures under way in the Research Laboratory of the Portland Cement Association. The tests were carried out in such a way that in addition to the direct comparison between slump and flow under the Standard A. S. T. M. procedure, various other comparisons were possible. In the flow test, readings were taken at 5, 10, 15 (standard) and 25 drops of the flow-table. Also the slump of the flow cone was measured just as in the slump test. This was done when the form was first removed and at the time of each successive reading of the flow. In addition, the specimen formed by the slump cone was treated in the same way, reading slumps and flow after the various number of drops. This gave an opportunity to observe the behavior of the concrete under the action of the flow-table for both types of specimens and the readings provide a numerical basis upon which the methods can be compared.

OUTLINE OF TEST

The cement used in this series of tests consisted of a mixture of equal parts of four brands of portland cement purchased in Chicago. The aggregate was calcareous sand and gravel from Elgin, Illinois. The sand was naturally graded 0-No. 4 and the gravel was No. 4-1½ graded as follows: 25 per cent No. 4-⅜ in., 50 per cent ⅜-¾ in., and 25 per cent ¾-1½ in. The concrete was mixed by hand in batches sufficiently large to fill one 6- by 12-in. cylinder. Two batches were made of identical mix. On the first batch, slump was measured in accordance with standard method. In this test the slump cone was supported on the 30-in. flow-table so that immediately following the measurement of the slump the table was lifted and dropped in exactly the manner used in the standard flow test, the height of drop being ½ in. Both the slump and the flow or the spread of the concrete cone were measured after dropping the concrete for 5, 10, 15 and 25 times. In this manner both the flattening and spread of the slump cone were obtained for different number of jiggings on the flow-table.

The next batch of identical concrete was placed in the standard flow cone and both flattening of the cone and the flow were taken for 0, 5, 10, 15 and 25 drops of the flow-table respectively. Each test was repeated on three different days and the averages used for this study.

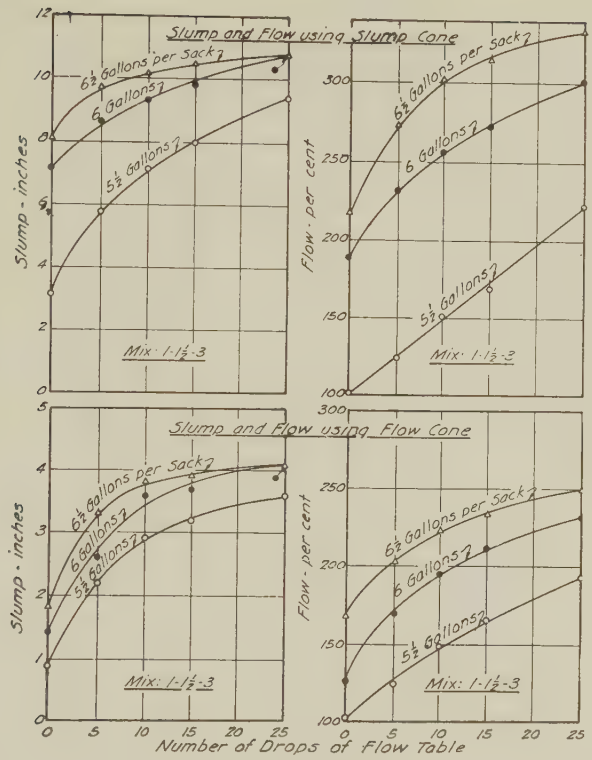


FIG. 1—EFFECT OF NUMBER OF ½-IN. DROPS OF FLOW-TABLE ON SLUMP AND FLOW OF CONCRETE

Five different concrete mixes were used: 1:1½:3, 1:2:3, 1:2:4, 1:2½:4 and 1:2½:5, by compacted volumes. Of these mixes the 1:1½:3, 1:2:4 and 1:2½:5 were repeated with the addition of 2, 5 and 10 per cent (by weight) of Celite, Pumicite and hydrated lime respectively, and with 2, 5, 10 and 20 per cent of portland cement as admixture. At least 3 different water-cement ratios were used for each mix, percentage and kind of admixture. The admixture was added to the dry materials before mixing.

TEST DATA

A complete set of test data is given in Tables 1 and 2. Table 1 gives the data obtained for slump and flow using the slump cone,

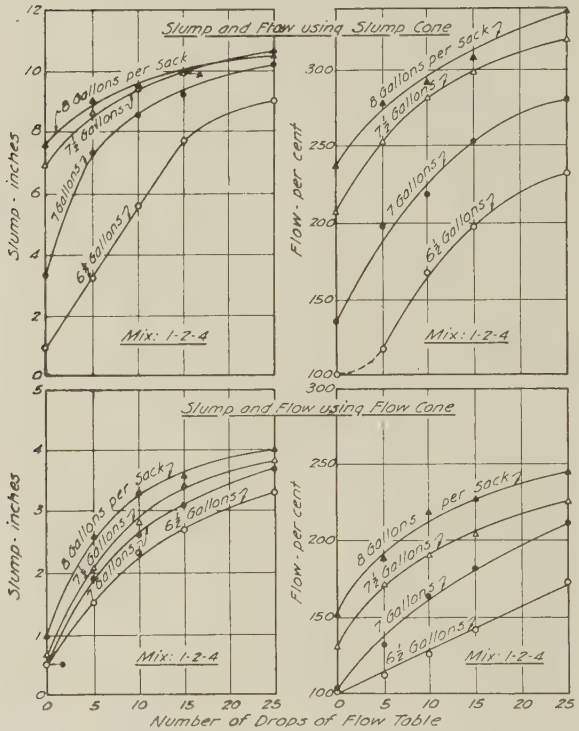


FIG. 2—EFFECT OF NUMBER OF ½-IN. DROPS OF FLOW-TABLE ON SLUMP AND FLOW OF CONCRETE

and Table 2 the corresponding values with the standard flow cone.

The effect of the number of ½-in. drops of the flow-table on slump and flow is shown in Figs. 1 to 3. The upper diagrams in these figures represent the values obtained with the slump cone while the lower diagrams show the corresponding values for the flow cone. Fig. 1 represents a concrete mix of 1:1½:3 by compacted volumes; Fig. 2, a 1:2:4 mix; and Fig. 3, a 1:2½:5 mix. The effect of an addition in water content on slump and flow is also seen from these figures.

Fig. 4 shows a comparison between flows obtained with the flow cone and with the slump cone for 5 drops of the flow-table.

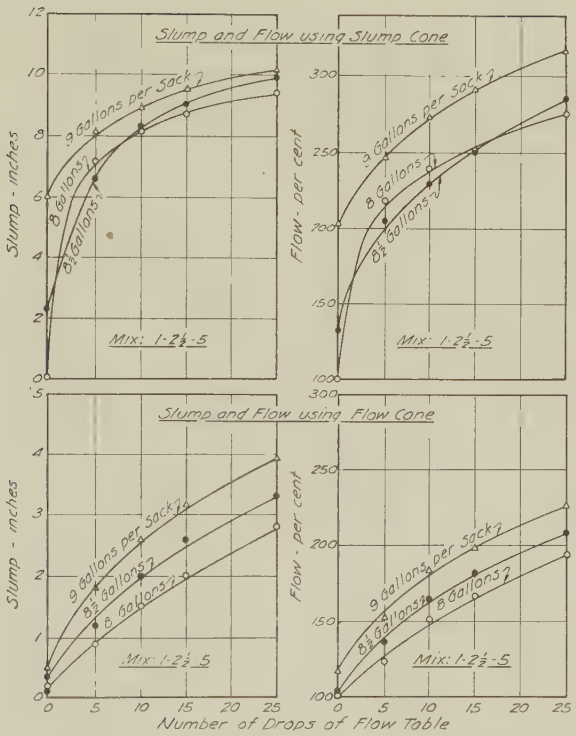


FIG. 3—EFFECT OF NUMBER OF ½-IN. DROPS OF FLOW-TABLE ON SLUMP AND FLOW OF CONCRETE

Figs. 5 to 8 give the relation between the standard methods of measuring slump and flow for plain concrete mixes and for concrete containing portland cement, Celite, Pumicite, and hydrated lime admixtures respectively, and Figs. 9 to 12 show a comparison between slump measured in the standard manner and flow measured with the slump cone after five ½-in. drops on the flow-table using concrete mixes containing the above admixtures. Fig. 12 and 13 give the effect of the admixture on both slump and flow of concrete for given water-cement ratios.

DISCUSSION OF DATA

Effect of Number of Drops of Flow-Table

Each ½-in. drop of the flow-table will shake the concrete to

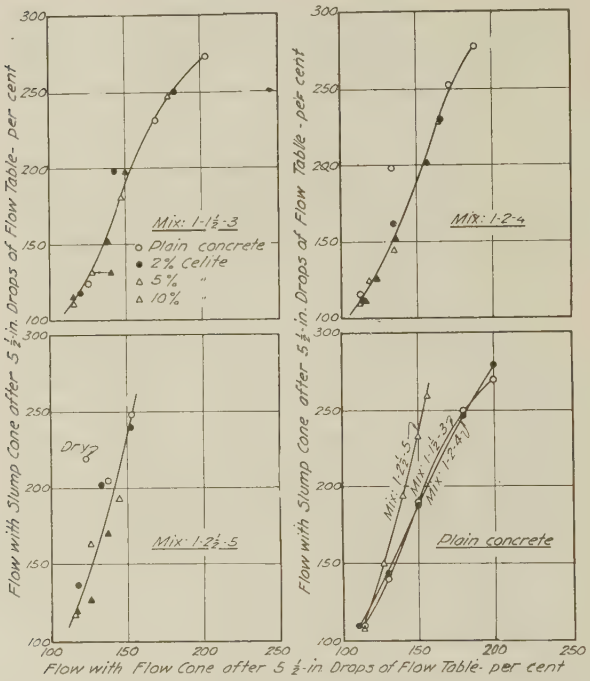


FIG. 4—RELATION BETWEEN FLOW OF CONCRETE USING FLOW CONE AND SLUMP CONE

such an extent that both the flattening and the spread of the cone will increase. Both slump and flow will therefore increase with the number of drops of the flow-table when using either cone as shown in Figs. 1 to 3. This increase will be rather uniform if the mix is fairly plastic and workable. With a harsh or very dry mix, a crumbling of the materials takes place, giving a sudden increase in both the slump and flow. This is more pronounced with the slump cone specimen. A typical example of the effect of crumbling on the slump and flow obtained with the slump cone is shown in Fig. 3 for 5 drops of the flow-table for a 1:2½:5 mix having 8 gal. of water per sack of cement. The crumbling of the cone during the shaking of the first few drops gave a large index of flow even though no actual flow of the concrete occurred. Slump

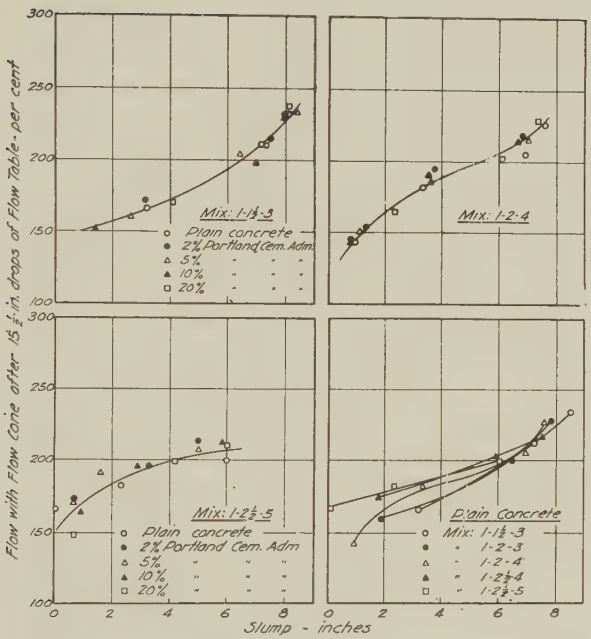


FIG. 5—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

or flow obtained on concrete which crumbles may be misleading when judging the consistency or placeability of concrete mixes.

In most cases the increase in slump and flow is greater for the first few drops of the flow table than for a larger number. Practically all the curves therefore have a steeper inclination for a small number than for a larger number of drops. This applies to both slump and flow with either cone. In the case, however, of the slump as measured on the slump cone, the curves also approach each other as the number of drops is increased. This means that the dry and stiff mixes show a larger increase in the slump with the increase in shaking of the concrete than do the wet and more plastic mixes. The sensitivity of the slump is therefore decreased with the number of drops. The resulting conclusion is that a slump test should be taken with no shaking

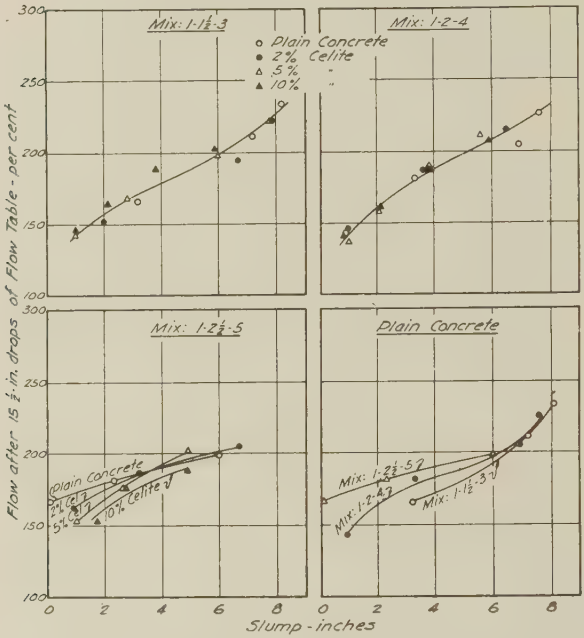


FIG. 6—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

of the flow table, which corresponds to the standard method of making the slump test.

The slumps measured with the flow cone, however, show a somewhat different behavior. The slumps on plastic mixes seem to increase with the increase in shaking just as fast and in some cases even faster than do the slumps for relatively stiff mixes. This difference in behavior is mostly due to the difference in shape of the concrete cone used, and indicates that for a flow cone the use of a large number of drops on the flow-table will not decrease the sensitivity of the slump materially.

The increase in number of drops of the flow-table seems to have little or no effect upon the sensitivity of the flow as measured either with the slump cone or flow cone. The curves remain

practically parallel to each other throughout the range used in this series so that no definite number of drops indicates any special advantage. It appears therefore of little importance at what number of drops the flow is taken; five drops seem to give just as sensitive a measure of the flowability of the concrete as the standardized 15 drops. Figs. 1 to 3 also show the effect of the water content on the consistency of concrete. In general the concretes of different water contents behaved similarly under increasing number of drops of the flow-table.

Slump Using Slump Cone and Flow Cone

The slump measured with the 12-in. slump cone of 8-in. base diameter is naturally much larger than the slump measured on the 5-in. flow cone of 10-in. base diameter. Figs. 1 to 3 show that the slump with the slump cone is usually several times as large as that with the flow cone for concrete of a given composition. The difference in slump for different water content is also correspondingly larger for the slump cone than for the flow cone. The slumps with the flow cone are actually so small that this method of test is practically useless for measuring plasticity of concrete. Just a glance at the curves in Figs. 1 to 3 shows the slump as measured on the standardized cone of the American Society for Testing Materials to be much more suitable than when measured on the flow cone.

Flow Using Slump Cone and Flow Cone

The flow with the slump cone is ordinarily much larger than with the flow cone for the same mix and number of drops. The difference in measured flow caused by changes in the water content is also considerably larger for the slump cone than for the flow cone. This is shown by Figs. 1 to 3 and leads to the conclusion that the slump cone provides a more sensitive test than the flow cone on the flow-table with ½-in. drops.

In Fig. 4 a direct comparison has been made between flows with the slump and flow cones using five ½-in. drops. The steep inclination of the curves indicates the much larger percentage of flow with the slump cone than with the flow cone. For the 1:1½:3 and 1:2:4 mixes which show an almost identical relation between flow with the two cones, a flow of 150 per cent on the flow cone corresponds to 190 per cent on the slump cone. This ratio re-

mains practically constant at all flows, indicating that the slump cone is a more sensitive measure of the changes in flowability of concrete than the flow cone. The addition of up to 10 per cent Celite by weight of cement does not seem to make any change in this relation. Neither of the two methods, therefore, shows any particular advantage over the other for measuring the smoothness of the mix produced by the addition of the Celite.

In the lower right hand corner of Fig. 4 each of the three plain concrete mixes is shown. It is noted that the 1:2½:5 mix has a considerably steeper inclination than the 1:1½:3 and 1:2:4 mixes. The relation between flow obtained with the flow cone and that obtained with the slump cone, therefore, varies somewhat with the richness of the mix. This affords a direct comparison between the suitability of each of these two methods as a measure of change in workability produced by changes in mix. For a given flow with the slump cone the leanest mix gives the lowest flow with the flow cone while for a given flow on the basis of the flow cone the richest mixes give the lowest flow with the slump cone. According to a general conception of workability of concrete a rich mix having the same flow on the standard flow test as a lean mix, will be the most workable of the two. Correspondingly, the flow cone will be selected as the one giving a better indication of real workability than the slump cone for use on the flow table. This fact partly offsets the advantage in the greater sensitivity of the slump cone found above.

Relation Between Standard Methods of Measuring Slump and Flow of Concrete

The relation between the two methods for measuring workability of concrete standardized by the American Society for Testing Materials is given in Figs. 5 to 9. Fig. 5 shows the relation for plain concrete mixes and concrete containing up to 20 per cent portland cement as an admixture. It is noted that the curve representing the relation between slump and flow changes somewhat with the richness of the mix. The addition of portland cement in the quantities used, however, is not large enough to make any material change in this relation. The two methods are therefore about equally suitable for measuring the effect of an

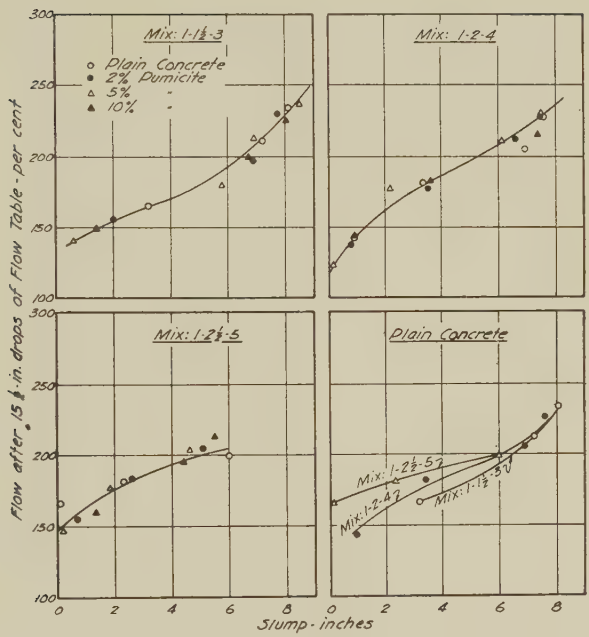


FIG. 7—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

addition of cement used as an admixture. For a mix as rich as 1:1½:3 the sensitivity of the flow as compared with the slump is not so large for dry consistencies as for wet consistencies. For leaner mixes, however, the relation is reversed and the sensitivity of the flow seems to be greatest for rather dry consistencies.

In the lower right hand corner of Fig. 5 the different mixes of plain concrete have been grouped together. This figure shows very clearly the variation in the relation between slump and flow caused by the richness of the concrete mix. It is generally considered that of two mixes having equal slump the richer mix is the more workable. A comparison between the 1:1½:3 mix and the 1:2½:5 mix shows that for the same slump the flow of the 1:1½:3 mix is considerably less than the flow of the 1:2½:5 mix. This is entirely contrary to a general conception of workability. If on

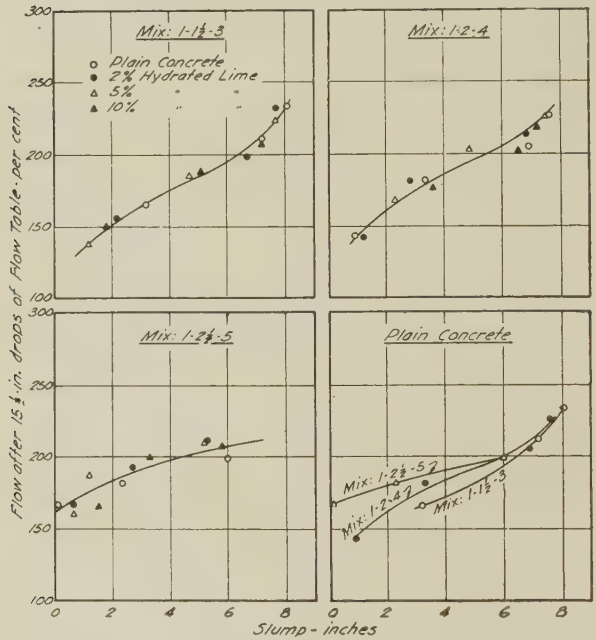


FIG. 8—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

the other hand a comparison is made on the basis of equal flow the richer mixes are seen to have the greatest slump. This strongly supports the conclusion that the slump is superior to the flow as an indication of the degree of workability of concrete mixes.

Fig. 6 shows the relation between the standard slump and flow when Celite is used as an admixture in amounts up to 10 per cent. The results indicate little or no change in the relation between slump and flow from the addition of Celite to 1:1½:3 and 1:2:4 mixes. The 1:2½:5 mix, however, was more affected by the Celite admixture; each increase in the amount of Celite gave a different relation between slump and flow. For a given flow the slump was increased with each addition of admixture. Thus the smoothing effect of the admixture changes the slump to a greater

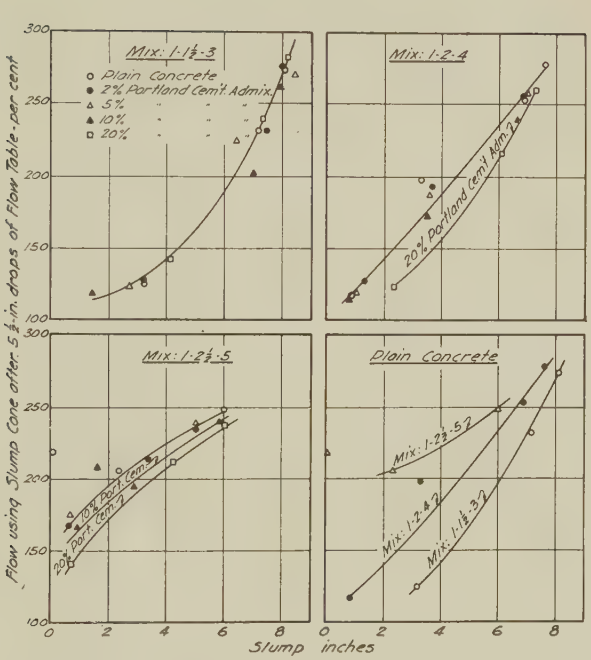


FIG. 9—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

extent than the flow so that the slump here also might be considered the more satisfactory measure of the workability of concrete.

Fig. 7 shows the relation between standard slump and flow for admixtures of Pumicite, and Fig. 8 for admixtures of hydrated lime. The effect of both these admixtures is seen to be very small for both the 1:1½:3 and 1:2:4 mixes, and even for the 1:2½:5mix the relation is practically unchanged. This indicates that either slump or flow is equally suitable for measuring the smoothing effect of these admixtures. This smoothing effect, therefore, appears to be less than that found by the Celite. This does not alter the conclusion drawn from other data obtained in

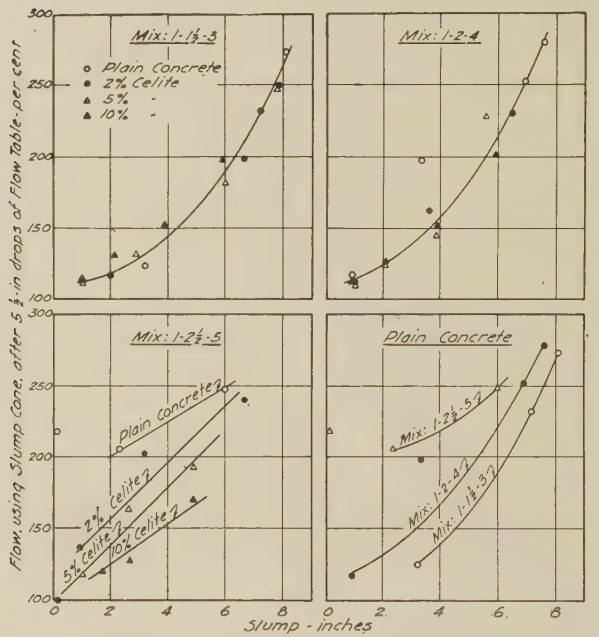


FIG. 10—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

this series of tests that the slump gives a more accurate indication of the real workability of a concrete mix than the flow.

Relation Between the Standard Slump Test and the Flow Using the Slump Cone After 5 Drops with Flow-Table

In the above discussion of the relation between flow measured with the flow and slump cones it was found that the slump cone gave the more sensitive measure, while the flow cone gave a somewhat better indication of real workability based on the richness of the mix. In view of the greater sensitivity shown by the flow when measured on the slump cone specimen, it was of interest to compare the results on this basis with those obtained by the standard slump test. This comparison is shown in Figs. 9 to 12 where the flow is based on 5 drops, using the slump cone specimen. These figures can be compared directly with Figs. 5 to 8 in which

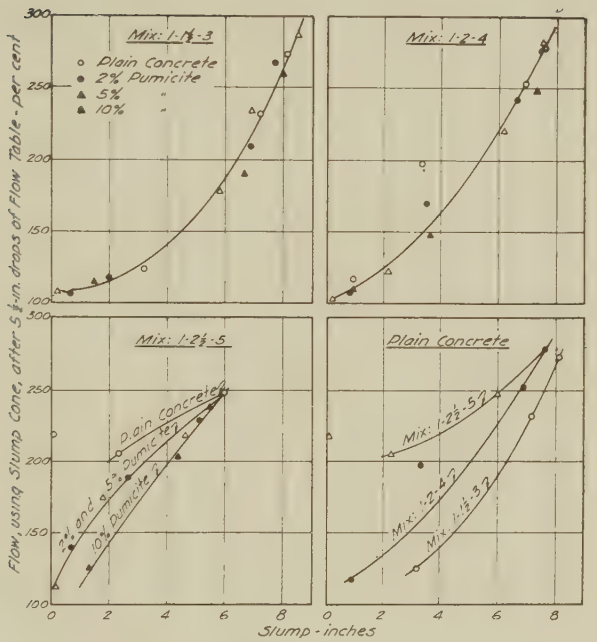


FIG. 11—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

the relation is based on the standard flow. In this comparison it will be seen that the curve in the pairs of figures: 5 and 9; 6 and 10; 7 and 11; and 8 and 12 are very similar in their general characteristics. They show two significant characteristics: First; for the rich mixes the flow is more sensitive than the slump in the case of the wet consistencies, but less sensitive in the case of the dry consistencies. Second; for the intermediate and lean mixes, the relationship is just reversed, the flow being less sensitive than the slump for the wetter mixes and more sensitive for the stiff mixes.

A further comparison of these pairs of figures brings out in an indirect way that the standard flow is a better measure of real workability than the flow using 5 drops with the slump cone specimen. This refers to improvement in workability, both by

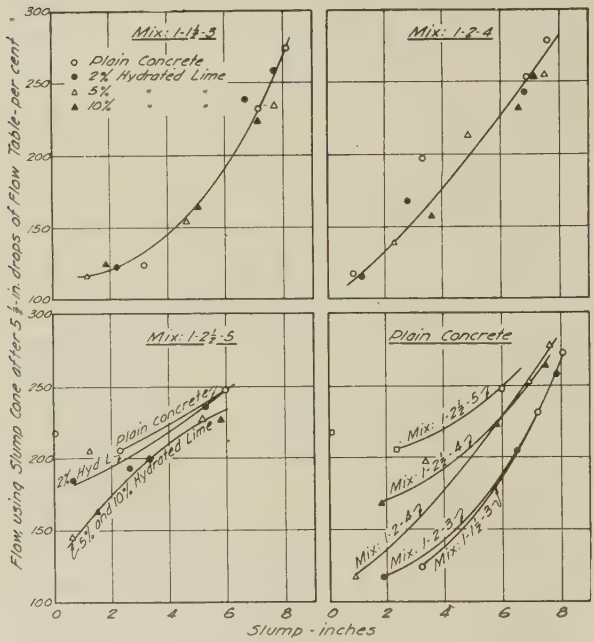


FIG. 12—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

increasing the cement content and by the use of some admixture which has a smoothing effect. In the discussion of Figs. 5 and 6, it was shown that for a given flow there was an increase in slump with an increase in richness for the plain concrete and an increase in slump with increasing percentages of Celite. This, it was pointed out, indicated the superiority of the slump as a measure of workability. In a study of Figs. 9 and 10 the same indication is seen in a much more marked degree, the slump being very plainly a better measure than the modified flow test of the increase in workability produced either by increasing the cement content or addition of Celite. Thus indirectly it is seen that in the flow test the standard flow cone gives a better indication of workability than the slump cone.

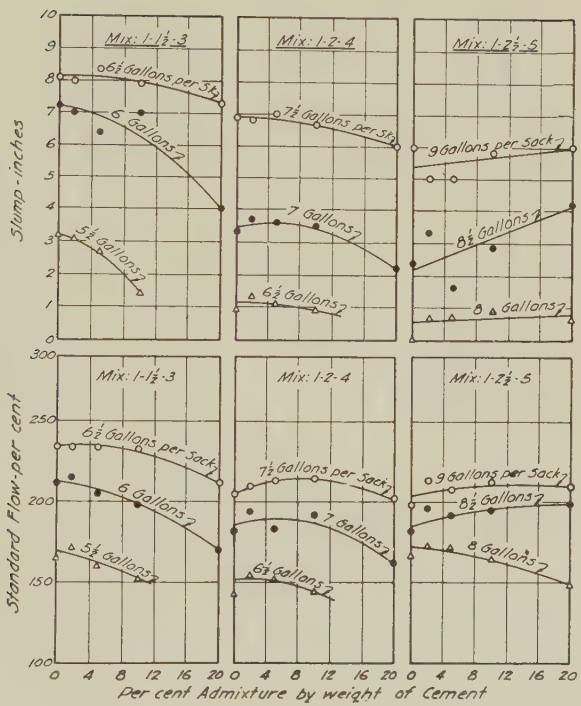


FIG. 13—EFFECT OF PORTLAND CEMENT AS AN ADMIXTURE ON SLUMP AND FLOW OF CONCRETE

In comparing Figs. 11 and 12 with Figs. 7 and 8, it is seen that in using the modified flow test, the relation between slump and flow is affected by additions of both Pumicite and hydrated lime, and also that the spread of the curves, representing the concretes without admixture, is very much greater. Both of these facts indicate the superiority of the slump test over the modified flow in bringing out the greater workability produced by enriching the mix or addition of admixture.

Effect of the Amount of Admixture on the Slump and Flow of Concrete

In the previous discussion it has been pointed out that the slump is a somewhat better measure than the flow of the improve-

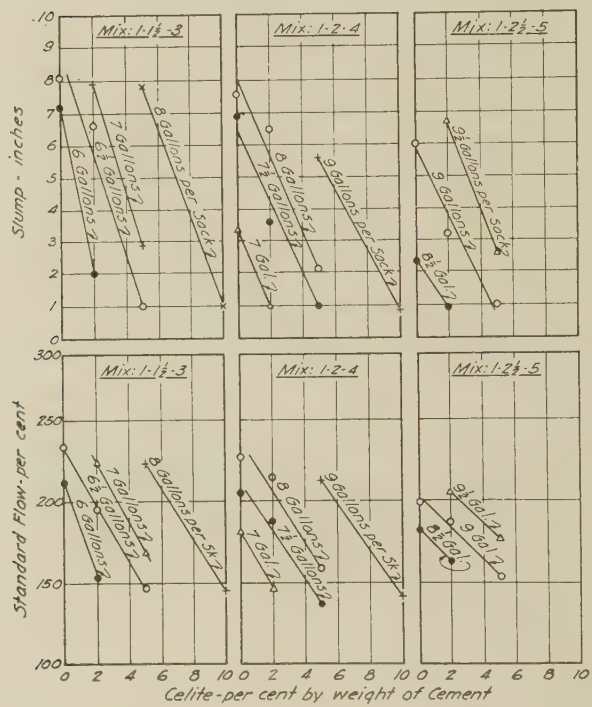


FIG. 14—EFFECT OF CELITE ON SLUMP AND FLOW OF CONCRETE

ment in smoothness resulting from the addition of admixtures to concrete. The actual effect of the amount of admixture on either the slump or flow, however, was not brought out. In Figs. 13 and 14 data are presented to show the effect of varying amounts of portland cement as an admixture and Celite on both the standard slump and standard flow for the several mixtures used in these tests. The data for the other admixtures are not shown as the curves for these lie between those for portland cement and Celite.

In Fig. 13 for portland cement as an admixture the upper diagrams contain the relations for the slump, and the lower ones those for the flow. It is noted that for a given water-cement ratio

(based on the original amount of cement) the slump is slightly decreased for the 1:1½:3 mix as the amount of cement is increased. A similar relation is seen to exist for the 1:2:4 mix but the decrease in slump is much less. The decrease in the slump for the 1:2:4 mix is negligible for additions of cement up to 10 per cent and only a slight decrease is noted for a 20 per cent addition. The 1:2½:5 mix, however, shows a different relation as the slump is somewhat increased with the increase in amount of cement. This indicates that the improvement on smoothness is not quite as great as the drying effect on rich mixes but greater than the drying effect on the lean mixes.

The results from the flow tests show a relation between the flow and amount of admixture for concrete of given water-cement ratio generally similar to that between the slump and amount of admixtures. The addition of cement decreases the flow of a rich concrete, but affects the leaner mixes to a lesser degree. For the 1:2½:5 mix the increase in flow with the addition of cement is slightly less than the increase in slump, indicating the previous finding that the slump is a more sensitive test than the flow for that smoother working which improves the workability of concrete.

In Fig. 14 the effect of Celite on a concrete mix of given water-cement ratio is shown. The slump as well as the flow is seen to decrease very sharply with the addition of the Celite. The upper diagram shows that for a 1:1½:3 mix of 6-gal. water ratio a slump of 7 in. in the plain concrete is reduced to 2 in. by the addition of 2 per cent Celite and an 8-in. slump in a 6½-gal. water ratio is reduced to 1 in. by adding 5 per cent Celite to the mix. To maintain a slump of about 7 in. in a 1:1½:3 mix the use of 2 per cent Celite requires the addition of ½ gal. of water per sack of cement and 6 per cent Celite requires an addition of 2 gal. over that required by concrete without admixture. The 1:2:4 and 1:2½:5 mixes show similar drops in slump with increase in percentage of Celite. The decrease in slump seems to be about proportional to the increase in percentage of admixture.

The relation between flow and per cent of Celite is shown in the lower diagrams of Fig. 14. Relations almost identical with those for the slump are found for the flow. The drying effect of

the admixture therefore affects both measures of consistency in a similar manner, and neither measure appears to be the better in comparing changes in consistency due to changes in water content. Fig. 6, however, showed the slump to be the more suitable measure of changes in workability due to admixtures, especially for the lean mixes. Fig. 5 (lower right hand diagram) also showed the slump test to be the better measure of increased workability due to increasing cement content.

The large difference between the curves in Figs. 13 and 14 is due to the kind of admixture used. While the addition of small percentages of cement did not change the consistency of a given mix to any important extent (water content unchanged) a very noticeable stiffening of the mix did result from the addition of corresponding percentages of Celite. Pumicite and hydrated lime did not show quite as great a drying effect as the Celite so that the relation between the slump and flow and the amount of these admixtures lies somewhere between the extremes represented in Figs. 13 and 14 for cement and Celite respectively.

SUMMARY

All data included in this series of tests show that the slump gives a more suitable indication than the flow of the increased workability of concrete produced either by enriching the mix or additions of admixtures. The results also indicate that the slump should be taken with no shaking of the concrete. The flow, however, remains about constant for the different number of drops on the flow-table so that any number is about equally suitable for the test. The use of five, ½-in. drops of the flow-table would seem to be as satisfactory as the use of 15, and would noticeably shorten the test.

The slump with the flow cone is not a sensitive test and is not suited to measuring consistency or workability.

The flow test is much more sensitive with the slump cone than with the standard flow cone for measuring changes in consistency due to changes in water content, also in showing the effect of increasing number of drops of the flow table. The use of the standard flow cone, however, seems to give a more accurate indication of increased smoothness resulting from richer mixes or the use of admixtures. This, therefore, partly offsets the ad-

vantage of the higher sensitivity obtained with the slump cone when used for measuring flow.

The relation between the flow and slump measured in accordance with the methods standardized by the American Society for Testing Materials changes with the richness of the concrete mix. For concrete of a given flow the slump increased as the richness increased, indicating that the slump is the better of the two methods for measuring improvement in workability obtained through increased cement content. The addition of admixtures also affects the slump to a greater extent than the flow, especially for lean mixes, which indicates further the superiority of the slump test over the flow test.

Modifying the flow test by using the slump cone instead of the standard flow cone increased the sensitivity of the test to changes in water content, but not to improvement in workability obtained through richer mixes or the use of admixtures.

While these tests have shown an increase in smoothness of concrete due to the use of admixtures they have also shown the stiffening of the mix which results from their use when no change is made in the water content.

(*See Tables 1 and 2 on following pages*).

Readers are referred to the JOURNAL *for May, 1931, for discussion of the two preceding papers, which may develop. Such discussion should reach the Secretary by April 1, 1931.*

A STUDY OF SLUMP AND FLOW OF CONCRETE

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE

Mix by volume: sufficient material used to make one 6- by 12-in. cylinder.

Cement: a mixture of equal parts of 4 brands of portland cement purchased in Chicago.

Aggregate (room dry when used):

Fine: sand from Elgin, Ill., graded from 0 - No. 4 (F. M. = 3.00).

Coarse: pebbles from Elgin, Ill., graded from No. 4 - 1½ in. (F. M. = 7.00).

Flow and slump tests made immediately after mixing concrete. A 4- by 8- by 12-in. slump cone was filled in 3 layers on a 30-in. flow table, each layer being rodded 25 times. Immediately after filling the cone was removed, both slump and flow recorded and the table was then given five ½-in. drops in about 3 seconds and the slump and flow again recorded. This operation was repeated on the same sample using 10, 15 and 25 ½-in. drops.

Each value is the average of 3 tests made on different days.

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
				Slump Cone--½-In. Drop									
1:1½:3	5½	——	—	3.2	5.8	7.2	8.0	9.4	101	124	151	169	222
	6			7.2	8.6	9.3	9.8	10.8	189	231	257	272	301
	6½			8.1	9.7	10.2	10.5	10.8	218	273	302	316	335
1:2:3	6	——	—	1.9	4.6	6.2	7.4	8.9	100	117	139	163	200
	6½			6.5	8.3	9.4	10.1	10.8	150	204	238	263	295
	7			7.8	9.4	10.0	10.5	11.0	208	259	288	306	331
1:2:4	6½	——	—	0.9	3.2	5.6	7.7	9.0	100	117	167	197	232
	7			3.3	7.3	8.5	9.2	10.2	135	198	219	252	280
	7½			6.9	8.6	9.4	9.9	10.7	207	253	281	298	320
	8			7.6	9.0	9.5	9.9	10.5	238	278	292	309	338
1:2½:4	7½	——	—	1.8	6.0	7.9	8.7	9.8	100	169	205	230	263
	8			5.8	8.2	9.5	10.3	10.8	175	222	254	274	309
	8½			7.5	9.1	9.6	10.0	10.7	220	264	292	308	329
1:2½:5	8	——	—	0.1	7.2	8.2	8.7	9.4	100	219	240	252	275
	8½			2.3	6.6	8.3	9.0	9.9	132	205	230	251	285
	9			6.0	8.1	8.9	9.5	10.2	203	248	273	291	318
1:1½:3	6	Celite	2	2.0	4.8	6.6	7.6	9.0	101	118	143	158	197
	6½			6.7	8.5	9.5	10.1	10.6	141	199	231	259	292
	7			7.9	9.6	10.3	10.6	10.9	188	250	283	303	328
1:1½:3	6½	Celite	5	1.0	3.5	5.1	6.4	8.1	100	111	127	136	169
	7			2.8	6.1	7.9	8.8	9.9	101	132	156	183	227
	7½			6.0	8.6	9.9	10.3	10.8	123	181	220	254	295
	8			7.8	9.4	10.3	10.5	10.8	183	248	289	303	331
1:1½:3	8	Celite	10	1.0	3.5	5.1	6.4	8.3	100	116	131	149	177
	8½			2.1	5.7	7.7	8.8	10.0	100	132	159	187	203
	9			3.8	7.7	9.2	10.1	10.8	105	153	192	222	268
	9½			5.9	8.9	10.1	10.4	10.7	129	199	221	266	269

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
Slump Cone—½-In Drop													
1:2:4	7	Celite	2	1.0	3.6	6.4	7.8	9.3	100	111	164	187	221
	7½			3.6	7.1	8.4	9.3	10.4	101	161	198	229	268
	8			6.5	8.4	9.6	10.2	10.5	180	230	264	282	313
1:2:4	7½	Celite	5	1.0	3.4	5.3	6.7	8.1	100	110	132	154	182
	8			2.1	5.3	6.8	7.8	9.2	100	125	153	178	217
	8½			3.8	7.0	8.5	9.3	10.2	102	145	186	220	265
	9			5.6	8.6	9.9	10.0	10.5	172	229	265	283	312
1:2:4	9	Celite	10	0.8	3.0	4.7	5.8	7.9	100	112	128	141	172
	9½			2.1	5.6	7.4	8.7	9.9	100	127	158	183	232
	10			3.9	7.5	9.2	9.9	10.6	102	152	193	224	270
	10½			5.9	8.6	9.8	10.5	10.9	134	201	243	272	305
1:2½:5	8½	Celite	2	0.9	5.3	7.3	8.4	9.5	100	137	177	208	244
	9			3.2	7.0	8.6	9.4	10.1	125	202	225	250	285
	9½			6.7	8.6	9.5	10.0	10.8	185	240	273	287	315
1:2½:5	9	Celite	5	1.0	4.0	6.2	8.1	9.7	100	118	157	186	221
	9½			2.6	6.4	8.4	9.4	10.1	100	163	206	232	272
	10			4.9	7.7	8.9	9.6	10.2	131	194	235	259	294
1:2½:5	10½	Celite	10	1.7	4.8	6.7	7.9	9.3	100	120	146	170	216
	11			2.6	6.1	7.9	9.2	10.3	100	127	168	199	248
	11½			4.9	8.0	9.4	10.1	11.0	102	170	218	253	292
1:1½:3	5½	Pumicite	2	2.0	4.7	6.5	7.5	8.9	100	118	139	164	200
	6			6.9	8.6	9.4	10.1	10.7	150	210	241	263	296
	6½			7.7	9.4	10.1	10.6	10.9	221	267	294	311	332
1:1½:3	5½	Pumicite	5	0.6	3.0	4.6	5.9	7.9	100	107	120	130	170
	6			5.8	8.1	9.2	9.9	10.5	129	179	214	243	278
	6½			6.9	8.8	9.6	10.4	10.7	186	234	263	283	309
	7			8.5	10.0	10.5	10.7	11.1	247	287	312	325	346
1:1½:3	6	Pumicite	10	1.4	4.0	6.0	7.0	8.4	100	115	131	150	181
	6½			6.7	8.6	9.8	10.3	10.7	137	190	229	255	293
	7			8.0	9.4	10.1	10.6	10.9	207	259	289	306	331
1:2:4	6½	Pumicite	2	0.8	2.6	5.3	7.0	8.6	100	109	133	152	187
	7			3.5	5.4	7.8	8.8	9.8	114	170	205	227	261
	7½			6.6	8.5	9.5	10.2	10.7	188	242	271	293	319
	8			7.5	9.1	9.7	10.3	10.8	235	277	296	315	328
1:2:4	6½	Pumicite	5	0.2	1.8	3.4	5.8	7.7	100	104	120	148	186
	7			2.2	5.2	7.0	8.0	9.5	100	121	172	199	243
	7½			6.2	7.9	8.9	9.6	10.4	173	220	252	273	303
	8			7.5	8.9	9.6	10.3	10.6	239	281	305	323	339
1:2:4	7	Pumicite	10	0.9	3.3	5.5	6.8	8.5	100	110	133	155	194
	7½			3.6	6.2	8.0	9.1	10.2	105	147	187	222	263
	8			7.3	9.3	10.0	10.6	11.0	198	249	277	299	326
1:2½:5	8	Pumicite	2	0.7	3.4	6.2	7.6	9.0	100	140	198	215	244
	8½			2.6	5.8	7.4	8.3	9.6	136	189	215	236	266
	9			5.1	7.7	8.8	9.4	10.3	191	229	256	279	306
1:2½:5	8	Pumicite	5	0.2	4.3	6.4	7.2	8.3	100	111	162	181	216
	8½			1.8	6.1	7.7	8.6	9.5	113	175	212	233	268
	9			4.6	7.0	8.3	9.0	9.9	176	219	246	269	298
1:2½:5	8½	Pumicite	10	1.3	4.7	6.7	7.9	8.9	100	126	183	209	243
	9			4.4	6.9	8.3	9.1	9.9	152	203	230	255	289
	9½			5.5	7.7	8.6	9.4	10.2	194	239	270	282	316

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
Slump Cone—½-In. Drop													
1:1½:3	5½	Portland	2	3.2	6.1	7.6	8.5	9.7	101	128	153	184	228
	6	Cement		7.5	9.0	9.5	10.1	10.8	184	231	262	283	312
	6½			8.0	9.4	10.3	10.5	10.9	233	276	301	318	341
1:1½:3	5½	Portland	5	2.7	5.5	7.1	8.2	9.3	101	122	146	167	207
	6	Cement		6.4	7.7	9.3	10.0	10.5	176	225	254	276	303
	6½			8.4	9.6	10.2	10.6	10.9	220	270	296	314	339
1:1½:3	5½	Portland	10	1.4	4.4	6.3	7.3	8.8	101	119	134	151	184
	6	Cement		7.0	8.6	9.4	9.8	10.6	153	203	236	259	292
	6½			7.9	9.5	10.4	10.4	11.0	215	261	289	307	328
1:1½:3	6	Portland	20	4.1	7.1	8.5	9.3	10.4	107	142	170	194	236
	6½	Cement		7.3	8.9	9.6	10.2	10.5	185	240	269	289	314
	7			8.2	9.7	10.4	10.6	11.0	233	282	308	325	344
1:2:4	6½	Portland	2	1.3	4.1	5.9	7.9	9.3	100	128	172	204	237
	7	Cement		3.7	7.3	8.6	9.3	10.1	130	194	225	255	285
	7½			6.8	8.4	9.4	10.0	10.6	203	255	281	300	325
1:2:4	6½	Portland	5	1.1	3.8	6.5	7.6	8.9	100	119	162	191	226
	7	Cement		3.6	6.8	8.2	8.9	9.7	125	188	219	241	277
	7½			7.0	8.7	9.6	10.3	10.7	209	258	284	302	328
1:2:4	6½	Portland	10	0.8	3.3	5.7	7.5	9.1	100	116	153	181	215
	7	Cement		3.5	6.8	8.6	9.4	10.6	119	172	207	231	272
	7½			6.7	8.8	9.7	10.2	10.9	186	240	270	290	316
1:2:4	7	Portland	20	2.3	5.3	7.0	8.1	9.4	100	123	156	182	223
	7½	Cement		6.1	8.3	9.4	9.9	10.7	163	217	247	270	300
	8			7.3	9.1	10.0	10.3	10.8	217	260	286	300	324
1:2½:5	8	Portland	2	0.7	4.5	7.2	8.2	9.3	103	168	211	232	258
	8½	Cement		3.3	6.8	8.1	9.1	10.2	173	213	235	193	295
	9			5.0	7.4	8.6	9.2	9.9	194	234	261	281	311
1:2½:5	8	Portland	5	0.7	5.3	7.9	8.9	9.8	100	174	203	224	260
	8½	Cement		1.6	6.4	8.3	9.0	9.9	148	209	241	265	292
	9			5.0	8.3	8.9	9.6	10.5	192	240	270	290	317
1:2½:5	8	Portland	10	0.9	4.8	6.9	8.0	9.1	100	167	207	221	256
	8½	Cement		2.8	6.2	7.9	8.7	9.9	133	195	221	250	280
	9			5.8	7.9	8.9	9.5	10.3	193	240	268	288	314
1:2½:5	8	Portland	20	0.7	4.7	7.3	8.3	9.6	100	140	189	211	241
	8½	Cement		4.2	7.1	8.5	9.2	10.1	152	211	241	264	291
	9			6.0	7.9	9.0	9.7	10.5	190	238	263	281	311
1:1½:3	5½	Hydrated	2	2.2	5.0	6.9	8.0	9.3	100	123	144	167	210
	6	Lime		6.7	8.6	9.6	10.1	10.7	182	238	272	287	315
	6½			7.7	9.2	9.9	10.4	10.9	195	256	287	309	334
1:1½:3	5½	Hydrated	5	1.2	3.6	5.3	6.5	8.1	100	117	130	148	181
	6	Lime		4.7	7.6	8.9	9.7	10.3	108	154	189	222	266
	6½			7.7	9.3	10.2	10.4	10.8	175	234	266	290	318
1:1½:3	6	Hydrated	10	1.8	4.7	6.7	7.7	9.2	100	125	142	158	197
	6½	Lime		5.1	7.9	9.0	9.6	10.4	117	165	198	228	268
	7			7.2	8.9	9.7	10.1	10.7	162	223	265	280	308
1:2:4	6½	Hydrated	2	1.2	3.9	6.2	7.7	9.4	100	115	163	190	228
	7	Lime		2.8	6.6	8.3	9.2	10.3	100	169	207	236	276
	7½			6.8	8.7	9.6	10.2	10.8	189	243	272	293	317
1:2:4	7	Hydrated	5	2.3	5.7	7.6	8.7	10.0	100	139	181	207	249
	7½	Lime		4.9	7.9	8.9	9.6	10.6	139	214	246	268	296
	8			7.5	9.2	9.9	10.5	10.9	191	254	288	305	331

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
Slump Cone—½-In. Drop													
1:2:4	7½	Hydrated Lime	10	3.6	7.4	8.7	9.5	10.2	100	158	198	229	267
	8			6.6	8.4	9.3	10.0	10.6	175	231	263	283	308
	8½			7.2	9.1	9.8	10.3	10.8	198	253	281	298	324
1:2½:5	8	Hydrated Lime	2	0.7	5.4	7.4	8.7	9.8	100	185	209	226	259
	8½			2.7	6.7	8.4	9.4	10.5	118	192	228	252	293
	9			5.3	7.6	8.7	9.5	10.4	191	238	265	286	320
1:2½:5	8	Hydrated Lime	5	0.6	5.2	7.4	8.8	9.8	100	146	185	206	237
	8½			1.2	6.9	8.2	9.1	10.2	102	205	231	255	286
	9			5.2	7.9	8.9	9.6	10.5	177	228	257	278	311
1:2½:5	8½	Hydrated Lime	10	1.5	5.5	8.3	9.1	10.1	100	164	203	227	264
	9			3.3	7.0	8.5	9.3	10.2	109	200	234	256	290
	9½			5.8	8.3	9.3	10.0	10.7	175	229	261	284	315

A STUDY OF SLUMP AND FLOW OF CONCRETE

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE

Mix by volume: sufficient material used to make one 6-by- 12-in. cylinder.

Cement: a mixture of equal parts of 4 brands of portland cement purchased in Chicago.

Aggregate (room dry when used):

Fine: sand from Elgin, Ill., graded from 0 - No. 4 (F. M. = 3.00).

Coarse: pebbles from Elgin, Ill., graded from No. 4 - 1½ in. (F. M. = 7.00).

Flow and slump tests made immediately after mixing concrete. A 6¾- by- 10- by- 5-in. flow cone was filled in 2 layers on a 30-in. flow-table, each layer being rodded 25 times. Immediately after filling the cone was removed both flow and slump recorded and the table was then given five ½-in. drops in about 3 seconds and the slump and flow again recorded. This operation was repeated on the same sample using 10, 15 and 25 ½-in. drops.

Each value is the average of 3 tests made on different days.

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
				Flow Cone—½-In. Drop									
1:1½:3	5½	——	0	0.9	2.2	2.9	3.2	3.6	102	125	149	166	193
	6			1.4	2.6	3.6	3.7	4.1	128	170	195	211	232
	6½			1.8	3.3	3.8	3.9	4.1	169	203	223	234	250
1:2:3	6	——	0	0.6	2.1	2.9	3.2	3.7	101	121	140	160	189
	6½			1.0	2.6	3.2	3.8	4.0	104	143	173	200	225
	7			1.4	3.1	3.7	3.9	4.1	148	189	218	229	248
1:2:4	6½	——	0	0.5	1.5	2.3	2.7	3.3	101	112	128	143	174
	7			0.5	1.9	2.6	3.1	3.7	102	133	164	181	211
	7½			0.7	2.1	2.8	3.4	3.8	132	171	190	205	226
	8			1.0	2.6	3.3	3.6	4.0	151	189	219	227	245
1:2½:4	7½	——	0	0.5	1.9	2.6	3.0	3.6	100	125	152	174	204
	8			0.9	2.2	3.0	3.5	3.9	107	154	182	203	227
	8½			0.9	2.6	3.4	3.8	3.9	130	176	201	216	238
1:2½:5	8	——	0	0.2	0.9	1.5	2.0	2.8	100	124	151	167	194
	8½			0.3	1.2	2.0	2.6	3.3	103	137	165	181	209
	9			0.5	1.8	2.6	3.2	3.9	118	153	184	199	227
1:1½:3	6	Celite	2	0.5	1.8	2.5	3.0	3.5	100	120	136	152	179
	6½			1.0	2.5	3.2	3.6	3.9	102	143	173	195	218
	7			1.5	3.1	3.7	3.9	4.0	133	181	207	223	243
1:1½:3	6½	Celite	5	0.3	1.6	2.2	2.8	3.3	100	116	129	143	164
	7			0.7	2.1	2.9	3.3	3.8	101	127	149	169	200
	7½			1.0	2.7	3.4	3.7	4.0	102	146	178	197	224
	8			1.4	3.0	3.5	3.9	4.1	126	178	204	222	244
1:1½:3	8	Celite	10	0.5	1.8	2.3	2.9	3.4	101	115	132	146	169
	8½			0.6	2.2	3.0	3.4	3.8	101	127	147	165	195
	9			0.9	2.6	3.3	3.6	4.0	103	138	168	189	217
	9½			0.9	2.9	3.5	3.9	4.0	104	150	183	204	230

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
Flow Cone—½-In Drop													
1:2:4	7	Celite	2	0.3	1.5	2.2	2.7	3.2	100	114	130	146	174
	7½			0.6	2.2	3.0	3.5	4.0	101	135	168	188	218
	8			1.0	2.8	3.5	3.7	4.0	113	166	196	215	236
1:2:4	7½	Celite	5	0.3	1.3	2.0	2.5	3.3	100	113	126	137	160
	8			0.6	1.8	2.8	3.0	3.6	100	119	140	159	191
	8½			0.9	2.4	3.2	3.5	3.9	102	136	168	190	217
	9			0.9	2.6	3.4	3.6	4.0	112	165	196	212	233
1:2:4	9	Celite	10	0.3	1.7	2.4	2.8	3.4	100	115	129	141	164
	9½			0.5	2.0	2.8	3.2	3.7	100	124	142	162	192
	10			0.9	2.5	3.3	3.7	3.9	102	137	164	187	218
	10½			0.9	2.8	3.5	3.9	4.1	105	158	189	209	235
1:2½:5	8½	Celite	2	0.4	1.5	2.4	2.9	3.6	100	118	139	162	195
	9			0.6	2.1	3.0	3.4	3.9	100	132	164	187	216
	9½			0.6	2.1	3.0	3.6	3.9	110	153	187	205	230
1:2½:5	9	Celite	5	0.4	1.6	2.2	2.9	3.6	100	116	133	153	185
	9½			0.6	2.0	3.3	3.3	3.9	100	127	155	177	209
	10			0.9	2.3	3.2	3.5	4.0	101	145	179	203	225
1:2½:5	10½	Celite	10	0.6	1.9	2.5	3.0	3.5	100	118	136	153	185
	11			0.6	2.0	2.9	3.4	3.9	101	127	153	177	209
	11½			0.8	2.6	3.3	3.6	3.9	101	138	170	189	221
1:1½:3	5½	Pumicite	2	0.5	1.9	2.7	3.1	3.6	100	120	138	156	187
	6			1.1	2.5	3.2	3.6	3.9	107	148	176	196	223
	6½			1.6	3.1	3.7	3.9	4.3	157	195	218	230	247
1:1½:3	5½	Pumicite	5	0.5	1.6	2.3	2.9	3.4	100	114	127	141	165
	6			0.9	2.2	3.3	3.4	4.0	102	131	156	180	211
	6½			1.6	2.7	3.2	3.7	4.1	135	168	190	214	236
	7			2.0	3.2	3.8	4.0	4.1	177	207	224	237	255
1:1½:3	6	Pumicite	10	0.4	1.7	2.5	3.0	3.6	101	118	134	150	178
	6½			1.2	2.7	3.3	3.7	4.0	110	151	178	200	224
	7			1.5	3.1	3.7	3.9	4.1	148	187	213	225	245
1:2:4	6½	Pumicite	2	0.4	1.5	2.0	2.5	3.0	100	111	125	137	160
	7			0.5	2.0	2.8	3.2	3.9	100	129	155	177	207
	7½			1.0	2.5	3.3	3.6	4.0	128	164	193	212	234
	8			1.7	3.1	3.7	3.9	4.2	158	194	216	229	249
1:2:4	6½	Pumicite	5	0.2	1.0	1.4	1.8	2.5	100	106	116	124	143
	7			0.5	2.0	2.9	3.4	3.9	100	123	153	178	209
	7½			0.9	2.6	3.3	3.7	4.1	122	162	191	212	234
	8			1.3	2.9	3.6	3.9	4.1	152	190	213	230	247
1:2:4	7	Pumicite	10	0.5	1.5	2.3	2.8	3.3	101	115	131	144	178
	7½			0.7	2.1	3.0	3.4	3.8	101	129	157	183	215
	8			1.1	2.8	3.5	3.9	4.2	128	167	198	216	239
1:2½:5	8	Pumicite	2	0.4	1.3	2.1	2.6	3.3	100	113	131	155	190
	8½			0.5	1.8	2.4	2.9	3.7	104	129	163	184	215
	9			0.7	2.2	2.8	3.4	3.9	118	155	184	206	231
1:2½:5	8	Pumicite	5	0.4	1.5	2.2	2.6	3.2	100	112	129	148	186
	8½			0.4	1.7	2.4	2.9	3.5	102	125	158	179	210
	9			0.7	2.4	3.1	3.6	4.1	110	158	186	206	236
1:2½:5	8½	Pumicite	10	0.5	1.8	2.5	3.0	3.6	100	115	138	160	196
	9			0.6	1.9	3.1	3.5	4.0	102	138	172	196	226
	9½			1.0	2.5	3.4	3.7	4.1	125	166	195	214	240

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
Flow Cone—½-In. Drop													
1:1½:3	5½	Portland	2	0.9	2.1	2.9	3.4	3.4	101	125	151	171	208
	6	Cement		1.3	2.9	3.6	3.9	4.1	133	172	198	216	238
	6½			1.7	3.4	3.8	4.0	4.2	160	199	219	234	251
1:1½:3	5½	Portland	5	0.7	2.0	2.8	3.1	3.7	100	121	140	160	190
	6	Cement		1.1	2.6	3.3	3.7	4.0	120	161	187	205	228
	6½			1.7	3.3	3.7	4.0	4.1	156	197	219	234	255
1:1½:3	5½	Portland	10	0.7	1.8	2.6	3.0	3.6	101	118	136	152	177
	6	Cement		1.1	2.6	3.3	3.7	4.0	111	151	178	199	223
	6½			1.8	3.3	3.9	4.0	4.3	153	192	216	232	250
1:1½:3	6	Portland	20	0.8	2.2	2.9	3.3	3.8	102	127	151	170	200
	6½	Cement		1.5	3.0	3.5	3.9	4.1	132	172	195	211	231
	7			1.8	3.4	3.8	4.0	4.3	166	205	227	239	256
1:2:4	6½	Portland	2	0.3	1.4	2.1	3.0	3.5	100	115	138	154	187
	7	Cement		0.7	2.2	3.0	3.6	4.0	104	140	169	195	224
	7½			0.9	2.7	3.5	3.8	4.1	131	176	203	219	243
1:2:4	6½	Portland	5	0.5	1.7	2.0	2.8	3.4	100	115	133	151	180
	7	Cement		0.7	2.1	2.9	3.3	3.9	101	131	160	183	213
	7½			0.9	2.5	3.3	3.8	4.0	130	167	193	213	238
1:2:4	6½	Portland	10	0.5	1.6	2.3	2.9	3.4	100	115	139	144	173
	7	Cement		0.9	2.2	3.0	3.5	4.0	101	137	168	191	221
	7½			1.0	2.8	3.5	3.8	4.1	129	171	195	215	238
1:2:4	7	Portland	20	0.6	1.9	2.7	3.1	3.6	100	122	145	164	198
	7½	Cement		1.0	2.6	3.1	3.7	4.0	110	152	182	202	229
	8			1.4	3.1	3.8	3.9	4.2	147	188	213	230	248
1:2½:5	8	Portland	2	0.5	1.4	2.4	2.9	3.6	100	121	146	173	205
	8½	Cement		0.5	2.0	2.6	3.1	3.9	112	146	176	196	225
	9			0.6	2.2	3.0	3.3	3.9	128	162	195	214	239
1:2½:5	8	Portland	5	0.5	1.6	2.3	2.8	3.5	100	116	146	171	205
	8½	Cement		0.5	1.7	2.5	3.1	3.8	104	139	170	191	222
	9			0.6	2.2	2.9	3.4	4.0	125	168	193	208	237
1:2½:5	8	Portland	10	0.5	1.8	2.6	3.0	3.8	100	117	143	164	198
	8½	Cement		0.6	1.9	2.7	3.3	3.9	106	139	173	195	222
	9			0.6	2.2	3.1	3.6	4.1	131	161	189	212	237
1:2½:5	8	Portland	20	0.3	1.4	2.1	2.6	3.2	100	112	132	148	182
	8½	Cement		0.6	2.2	2.9	3.4	4.0	112	145	176	199	228
	9			0.9	2.5	3.3	3.6	4.0	117	159	187	210	233
1:1½:3	5½	Hydrated	2	0.6	2.0	2.7	3.1	3.6	100	120	138	156	185
	6	Lime		1.0	2.5	3.2	3.6	4.0	111	150	179	199	228
	6½			1.3	3.1	3.7	3.9	4.1	147	190	214	231	252
1:1½:3	5½	Hydrated	5	0.4	1.5	2.2	2.6	3.2	100	115	127	138	164
	6	Lime		0.8	2.0	3.1	3.5	3.9	102	136	164	185	213
	6½			1.0	2.9	3.6	3.9	4.1	129	177	206	223	246
1:1½:3	6	Hydrated	10	0.6	1.9	2.5	3.0	3.5	100	120	135	150	175
	6½	Lime		1.0	2.5	3.2	3.5	4.0	103	138	166	188	214
	7			1.2	2.8	3.5	3.9	4.0	119	163	190	207	229
1:2:4	6½	Hydrated	2	0.3	1.3	2.0	2.2	3.2	100	111	125	141	170
	7	Lime		0.6	2.0	2.9	3.3	3.9	100	125	155	181	219
	7½			0.9	2.6	3.3	3.8	4.0	129	169	197	214	236
1:2:4	7	Hydrated	5	0.5	1.9	2.8	3.3	3.8	101	119	146	169	202
	7½	Lime		0.9	2.5	3.3	3.7	4.0	107	150	185	204	229
	8			1.2	3.0	3.6	3.9	4.1	132	182	210	227	248

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
Flow Cone—½-In. Drop													
1:2:4	7½	Hydrated Lime	10	0.7	2.2	2.9	3.3	3.9	101	128	158	177	207
	8			0.9	2.7	3.5	3.7	4.0	106	156	188	202	229
	8½			1.2	2.9	3.5	3.9	4.0	130	176	204	218	239
1:2½:5	8	Hydrated Lime	2	0.5	1.8	2.5	3.1	3.5	100	120	146	166	199
	8½			0.5	1.9	2.8	3.4	4.0	101	131	164	193	225
	9			0.7	2.2	3.1	3.6	4.0	124	153	190	211	238
1:2½:5	8	Hydrated Lime	5	0.4	1.5	2.3	2.8	3.5	100	115	137	160	196
	8½			0.6	2.1	3.0	3.4	3.9	100	130	161	187	219
	9			0.5	2.3	3.1	3.5	4.0	115	157	188	210	231
1:2½:5	8½	Hydrated Lime	10	0.6	1.9	2.7	3.2	3.9	100	122	147	165	201
	9			0.7	2.3	3.1	3.6	4.0	100	140	177	200	227
	9½			0.7	2.4	3.3	3.6	4.1	111	155	188	209	236

Current Researches on Plain and Reinforced Concrete and Related Materials

Report of Committee 101, Survey of Current Research

H. F. GONNERMAN, CHAIRMAN

INTRODUCTION

With the reorganization of the standing committees of the American Concrete Institute, Committee E-3, Research, was replaced by Committee 101, Survey of Current Research. With the change in the committee's name, a change was also made in its function. Those members of the Institute who are familiar with the reports of the old committee will recall that about half of the report was devoted to brief abstracts on researches published in current American and foreign engineering journals. The interest shown in these brief abstracts was one of the considerations, when Journal publication was inaugurated, in the creation of a section of Abstracts, and the formation of the new Committee on Abstracts, whose efforts would be entirely devoted to this type of work, giving much greater scope to the undertaking. Committee 101 will no longer include bibliographies in its report, but will devote its time entirely to assembling and compiling information on researches received from various college, state and commercial laboratories throughout the country. To this end, the subject-matter of this report has been prepared and is arranged in the following sections:

1—*Researches on cement.* Most of the researches already carried out or now in progress on cement are concerned with the constitution of portland cement and means of improving it, as well as with tests on high early strength cements. In addition, reference is made to the development of a comparatively inexpensive damp closet for cement testing.

2—*Researches on aggregates.* This section includes tests on durability of aggregates, effect of size and shape of aggregate particles, effect of heating aggregates prior to their incorporation in the mix, effect of grading, and the development of a comparative strength test for sand.

3—*Researches on plain concrete.* In the field of plain concrete the trend of the investigations is toward comparative tests on methods of curing, durability of concrete under various conditions of natural exposure, effect of use of various admixtures, and factors affecting volume changes. In addition, short summaries are given on tests of bricklaying problems, fatigue, flow under sustained stress, jigging, mixer operation, modulus of elasticity and Poisson's ratio, pavements, permeability and methods of obtaining watertightness, cast stone, compressive and flexural strength, temperature effects, uniformity of concrete test methods, wear, and workability.

4—*Researches on reinforced concrete.* The work on reinforced concrete includes studies on bond resistance of reinforced concrete beams under continued load, tests on bridge arches, and studies on type of reinforcing. An important investigation on columns is being carried out under the auspices of Committee 105 of the American Concrete Institute.

5—*Suggested researches on concrete and related subjects.* The number of researches suggested as needed and of prime importance indicates that there is a never-ending source of problems requiring study in the efficient use of concrete. These will be of particular interest to laboratories seeking subjects on which to direct their efforts.

This report has been submitted to letter ballot of the Committee, which consists of 11 members, of whom seven have voted affirmatively, none negatively, and four have refrained from voting.

H. F. GONNERMAN, *Chairman.*
F. E. RICHART, *Secretary.*

(NOTE—Many of the researches listed have not been completed, and in general, reports giving data of the tests are not available. In a few instances reference is made to the publication of reports. The conclusions given for certain of the investigations are those of the investigator and do not necessarily represent the views of the Committee.)

1—Cement

Improvement of Portland Cement: (Board of Water Supply, New York City). Studies in progress since 1914 with a view to improving the quality of portland cement, have covered such subjects as: Solubility in sugar solutions, heat of setting, alkali content, resistance to sodium sulfate solutions, lime-silica ratios, tests of cements previously used, and effects of calcium chloride and other salts. Results from these investigations are published from time to time in various engineering journals.

Stabilization of Di-Calcium Silicate: (Ohio State University, Columbus). Studies were made in the Department of Mineralogy on the properties of di-calcium silicate, particularly as they relate to refractories, but also in their application to portland cement products in the question of dusting. The results will be published at an early date by the Engineering Experiment Station.

The Upper SO_3 Limit in Portland Cement: (Mellon Institute, University of Pittsburgh). Only enough SO_3 should be added to portland cement to control its initial set, owing to the crystallizing tendency of the sulfoaluminate formed. Compressive strengths of small specimens of 1:3 mix, made from high early strength cement flour and Ottawa standard sand were about 5500 pounds per square inch at 24 hours and then rose steadily to 9200 at one year. As the great bulk of the cement was hydrated within the first day the increase strength must be ascribed to the induration reactions occurring within the silica gel.

Specifications and Tests for High Early Strength Cements: (State of Washington, Department of Highways, Olympia). The purpose of this investigation is to develop suitable specifications for the use of high-early strength cements. Tests were made on various kinds of cements at various ages using 6 by 12-in. cylinders, beams, and small mortar specimens. The data obtained thus far indicate that standard Ottawa sand briquets are unsatisfactory test specimens.

Development of a Low-Priced Damp Closet for Cement Testing: (Lehigh Portland Cement Co., Allentown, Pa.). Urgent need of an efficient low-priced damp closet for cement testing has led to a rather extensive search for simple devices for control of temperature and humidity. The most promising combination so far developed consists of a household refrigerator of the "top-icer" type, modified as to interior construction to provide a storage tank and shelves for pats and briquets; a nozzle of the oil spray type for discharging a water spray to provide humidification close to saturation, temperature control, and continuous circulation of the air inside the closet; and a thermostatic element to control the temperature of the water supply to the spray. While the limitations of this combination are not yet fully established, the object in view is to provide a cabinet that will automatically insure the conditions prescribed by the cement specification at all seasons of the year at a cost not to exceed $200 for the complete installation.

Slide-Rule for Calculating Compounds in Portland Cement: (Lehigh Portland Cement Co., Allentown, Pa.). In an effort to simplify the computation

of compound composition of portland cement, it was found that the percentage of any compound can be calculated directly from the analysis without the previous calculation of the percentage of any other compound. The series of equations for direct computation have been used as a basis for the design of a slide-rule for calculating the compound composition of portland cement. The slide-rule is described and illustrated in an article by L. A. Dahl, "A Slide-Rule for Calculating Compounds in Portland Cement," *Rock Products*, November 9, 1929, p. 50.

High Early Strength Concrete: (Pennsylvania Department of Highways, Harrisburg). This investigation was undertaken to determine the efficiency of high early strength cement compared with blended standard portland cement. Cement-sand mortar and concrete beams and cylinders were molded for test at age of 1 day to 1 year. Tension and compression tests were made on the cement sand mortars, beams were tested for modulus of rupture, and cylinders were tested in compression. The special cement showed increased strength at ages of 1 to 28 days, but at the 3-month period the strengths were nearly equal in the mortar tests and in the modulus of rupture tests on concrete. The concrete compression tests showed the special cements to be slightly higher than the standard portland.

In another series of tests which has just been completed, a high-early strength cement showed a considerable gain in mortar tension and compression for the first 7 days. At the 28-day period and later periods, the strengths converged, and in some cases the standard cement showed slightly higher strengths. In the concrete tests the high early strength cement showed slightly higher strengths at the 1- and 2-day periods, but strengths thereafter were about equal to those of the standard portland cement.

Comparative Strength Tests of Cement: (Iowa State Highway Commission, Dept. of Materials and Tests, Ames). Tests were carried out in cooperation with Sub-Committee VII of A. S. T. M. Committee C-1 on Cement and the Special Committee on Cement of the American Society of Civil Engineers, and the results obtained were forwarded to the respective committees for comparison with the results obtained by the other cooperating laboratories.

Study of the Effect of Quality of Cement on Quality of Concrete: (U. S. Bureau of Public Roads, Washington, D. C.). 3-year tests have been completed and report will be issued shortly.

Researches on Cement: (Portland Cement Association Fellowship, National Bureau of Standards, Washington, D. C.) The investigations of this Fellowship are designed to explore the major chemical problems of the cement industry. Among the investigations now in progress are the following:

(1) A study, with pure materials, of the influence of composition on the volume constancy of specimens of the set cements. The compositions examined include the cement compounds, mixtures of compounds, laboratory cements of systematically varying composition prepared under definitely controlled conditions, and commercial cements. The specimens, 6 by 1 by 1-in. are stored (a) continuously in air, (b) continuously in water after the

first 24 hours, and (c) alternately in air and in water at 7-day intervals. Length measurements are made at definite time intervals, over a period of years, using a microscope comparator and a micrometer comparator.

(2) A comprehensive study of the set of cements which includes research on (1), the chemical phenomenon of set; (2), the influence of rate of set on the properties of the cement; (3) the measurement of rate of set; and (4) the control of set. During the past year some of the causes have been examined of the so-called "false" or "hesitating" set. Also, a study is under way on the heat of setting and a correlation of the heat evolved with the reactions taking place as determined by microscopic, chemical and physical tests.

(3) A fundamental investigation on the kinetics and thermodynamics of the hydration or setting process. The rate and degree of hydration are being measured by observations on the change in density of samples of the cement compounds of known specific surface, and by observations on the heat of hydration of the compounds. Likewise, a thermal analysis of the hydrated compounds is being made by the continuous weighing of samples undergoing dehydration under rigidly controlled conditions.

(4) Phase equilibria study of the cement systems. During the past year the system CaO-Al_2O_3-Na_2O has been studied. Soda is commonly present in cements in small amounts and it is important to have information on the compounds which it forms. Only then can a rational study be made of the influence of the soda under all conditions.

(5) The development of the x-ray method to a study of the constitution of portland cement. It often is not possible by microscopic or chemical means to identify the phases present in laboratory or commercial cements. The x-ray method has been developed to a point where it is of great value in the identification of compounds and in the following of reactions. Many of the problems which have long baffled investigators have yielded to the convincing criteria of the x-ray.

Recent publications of the Fellowship are as follows:

Paper No. 21—"Calculation of Compounds in Portland Cement," by R. H. Bogue, *Ind. Eng. Chem.* (Analytical Edition), I, (1929), 192; *Rock Products 32*, No. 23 (1929), p. 47.

Paper No. 22—"The Influence of Magnesia, Ferric Oxide and Soda upon the Temperature of Liquid Formation in Certain Portland Cement Mixtures," by W. C. Hansen. *Bureau of Standards J. of Research*, Jan., 1930, Research Paper No. 132.

Paper No. 23—"Revised Procedure for the Determination of Uncombined Lime in Portland Cement," by William Lerch and R. H. Bogue. *Ind. Eng. Chem.* (Analytical Edition) *2* (1930), 296; *Concrete, 37*, (1930), 88.

Paper No. 24—"The X-ray Method Applied to a Study of the Constitution of Portland Cement," by L. T. Brownmiller and R. H. Bogue. *Bureau of Standards J. of Research*, October (1930) R. P. No. 233, *Am. J. Sci.* (in press).

Researches in Cement: (A. S. T. M. Committee C-1 on Cement). At the meeting of Committee C-1 of the American Society for Testing Materials held in Washington, Sept. 6, 1930, the following sub-committees were formed:

"A"—Methods of Testing
"B"—Specifications for Portland Cement
"C"—Specifications for High Early Strength Cement
"D"—Specifications for Masonry Cement

These sub-committees are empowered to form technical committees for the purpose of studying specific problems. The technical committees are not to be considered as standing committees but are expected to complete their work and make a final report as early as possible.

Sub-Committee "A" will give its immediate attention to the rewriting of the Manual and will undertake to suggest such changes in present apparatus as may be necessary and also to develop better apparatus for cement testing than is available at present. The development of a moist closet which will maintain constant temperature and humidity is quite an urgent question as many laboratories are willing to install this equipment if a suitable one can be economically manufactured.

Sub-Committee B has appointed 3 technical committees to report on the following problems relating to portland cement:

1. Relation of Particle Size to Physical Properties,
2. Relation of Chemical Composition, particularly as Affected by Ratios, to Physical Properties,
3. Study of Accelerators and Retarders,
4. Effect of Temperatures at time of use,
5. Study of Accumulated Data on Plastic Mortar Strength Tests, and Recommendation as to Whether Work Shall be continued on this Subject,
6. Study of Tests Indicative of Durability,
7. Early-age Tests.
8. Factors Affecting Volume Change.

2—AGGREGATES

Effect of Mixing Coarse Screenings with Sand which Does Not Meet Specifications: (Kentucky State Highway Department and University of Kentucky, Lexington). This procedure has been in operation for two years, with excellent results. A complete report on the findings from this study will be published soon.

Durability of Concrete Made from Unsound Stone: (Kentucky State Highway Department and University of Kentucky, Lexington). This investigation included tests on concrete made with 50 and 100 per cent unsound fine and coarse aggregate. Concrete made with 100 per cent unsound aggregate and exposed to weather tested as follows: 3 per cent increase in strength from 28 days to 6 months, with no failures due to disintegration; 11 per cent decrease in strength from 6 months to 1 year, one specimen in three having failed through

disintegration; 44 per cent increase in strength from 1 year to 2 years, with two failures due to disintegration.

Concrete made with only 50 per cent unsound aggregate tested as follows: 36 per cent increase in strength from 28 days to 6 months; 13 per cent increase from 6 months to 1 year; 27 per cent increase from 1 to 2 years. In this group of tests there was some crazing but no disintegration. Tests will be made through the 5-year period.

Sandstone as Aggregate in Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). An extensive investigation has been in progress since 1920 to determine the suitability of sandstone as fine and coarse aggregate in the construction of concrete roads and structures. It was found that any sound sandstone having a percentage of wear of less than 7 can be used in concrete pavement under proper control methods. Sandstone with a percentage of wear as high as 15 can be used for structures. A report is being prepared for early publication.

Method of Testing Stability of Sandstone for First-Class Masonry Construction: (Kentucky State Highway Department and University of Kentucky, Lexington). Tests were made to determine the reliability of sodium sulfate soundness tests on sandstone common to Kentucky. Specimens which went to pieces under the accelerated test have not been affected after two years in the weather, and the same stone was not affected by twenty alternations of freezing and thawing. Complete chemical analyses are being made of the stone which passed the accelerated soundness test and on the samples which did not. Though indications are that this test is not suitable for this type of aggregate, no conclusions can be drawn until further tests are made.

Properties of Concrete Made with Light-Weight Aggregate: (T. & A. M. Dept., University of Illinois, Urbana). The purpose of this investigation was to determine the properties of a structural grade of Haydite concrete such as might be suitable for reinforced concrete buildings and other structures. Studies have been made thus far on the properties of the aggregate, as well as on compressive strength, weight, density, modulus of elasticity, and workability of concrete of varying proportions, richnesses, and consistencies. In addition, determinations are being made of the resistance to diagonal tension, bond and compressive stress in flexure of reinforced concrete beams. Data of the tests were given in paper on "Tests of Plain and Reinforced Haydite Concrete," by F. E. Richart and V. P. Jensen, Proceedings, American Society for Testing Materials, Vol. 29, 1930; also by the same authors, "Construction and Design Features of Haydite Concrete," this JOURNAL, A. C. I., Oct., 1930, p. 151; "Tests of Bonding Floor Finish to Slabs of Haydite and Sand Concrete," JOURNAL, A. C. I., Dec. 1930, p. 339.

Effect on Strength of Soft Particles of Coarse Aggregate on the Strength and Durability of Concrete. (Michigan State Highway Department Laboratory, Lansing). This investigation was undertaken to determine to what extent the strength of concrete is affected by the presence of shale, friable sandstone, ochre, etc. 6 by 12-in. cylinders and 6 by 6 by 40-in. beams were made from concrete in which an aggregate known to be sound was mixed with varying

percentages of soft and non-durable particles. In one group of tests, the specimens were broken after 28 days' moist curing, and in another group after 25 cycles of freezing and thawing following 28 days moist curing. The strength of concrete, both moist-cured and subjected to freezing and thawing, was reduced by the presence of these particles. Report on this investigation by W. J. Emmons published, Proceedings of the 16th Annual Conference on Highway Engineering, University of Michigan, p. 45.

Effect of Character of Fine Aggregate on Mortar Strength: (Minnesota Highway Department, St. Paul). Six sands from different pits in Minnesota were artificially graded and tested in mortar in compression, tension, and flexure. The results show that with grading and water-cement ratio constant, variation in shape and characteristics of the sands had practically no effect on the strength of the resultant mortar.

Effect of Heating Aggregates on Concrete Strength: (Minnesota Highway Department, St. Paul). Compression and transverse specimens made for test at 3, 7, 14, 28 and 180 days, 12, 18 and 24 mo., from sand, gravel, and water heated sufficiently to bring the temperature of the concrete to 70, 100 and 130 degrees F. when deposited in the molds. Time of set determined by flow. It appears from the data obtained thus far that: (1) heated concrete requires more water to produce a given workability, (2) there is a reduction in strength so great that the difference in water-cement ratio can hardly be the cause, (3) heated concrete sets more quickly than unheated, (4) concrete should be placed in forms at a temperature of not over 70 to 80 degrees F.

Joint Influence of Iron and Aluminum in Native Sands and Mortar Strengths: (Maine Technology Experiment Station, Orono). The status of this investigation was reported in the Proc. of National Acad. of Sciences, v. 15, p. 742, Sept., 1929, and reprinted as Paper 9 of the Maine Technology Experiment Station.

Influence of Aluminum on Mortar Strength: (Maine Technology Experiment Station, Orono). The status of this investigation was reported in the Proceedings of the National Academy of Sciences, v. 15, p. 740, Sept., 1929, and reprinted as Paper 8 of the Maine Technology Experiment Station.

Study of Hydrogen Ion Content of Maine Sands in Relation to Their Use in Concrete: (Maine Technology Experiment Station, Orono). It appears from the results obtained thus far that the hydrogen ion content will add much to the knowledge of strength-making qualities of sands.

Selection of Mortar and Plaster Sands: (Lehigh Portland Cement Company, Allentown, Pa.) Two comparatively new methods of test were employed in these studies, one of which is the so-called plastic mortar test which provides a means of judging sands from the point of view of economy, as well as a method for determining quantitatively the deleterious effects of organic matter in sands. The other was a simple titration method for determining the absorption of sand. The former test is under investigation by Committee C-1, of the American Society for Testing Materials as a possible specification test for portland cement and the latter test is useful in connection with all concrete tests in which the absorption of the fine aggregate is a factor. The

sand tests were described and discussed by J. C. Pearson in paper on "Fine Aggregate in Mortar and Plaster" (see Proc. A. S. T. M. 1929) and "A Simple Titration Method for Determining the Absorption of Fine Aggregate" (see *Rock Products*, p. 64, May 11, 1929).

The Laws of Grading Derived and Applied to Aggregates and Cement: (Mellon Institute, University of Pittsburgh and Bureau of Mines, Minneapolis Station). Fundamental laws have been derived by a mathematical analysis for maximum packing, giving two systems, one with an intermittent or gap grading and the other continuous. These have been applied to mortar, where noteworthy applications have been made to stucco, cast stone and masonry mortars, to concrete and to the proper distribution for the cement itself. The function of admixtures is analyzed.

Effect of Grading of Fine and Coarse Aggregate on Strength of Concrete: (National Sand and Gravel Association, Washington). This investigation was designed to furnish information on the effect of maximum size of sand and gravel on quality of concrete, and the effect of variations in grading within the usual specification limits. Tests were made in compression and flexure, using three different concrete mixes involving three gradings of fine aggregate and seven gradings of coarse aggregate.

Effect of Flat Particles on Concrete-Making Properties of Gravel: (National Sand and Gravel Association, Washington). The report on this investigation, which consisted of compressive and flexural tests of concrete of two different mixes using gravel free from flat particles and gravel containing 15 per cent flat particles, was published in the 1929 Proceedings of the American Society for Testing Materials, and in the July, 1929, issue of the National Sand and Gravel Bulletin.

Effect of Grading on Void Content: (National Sand and Gravel Association, Washington). This investigation supplements the work on this subject described in the 1928 Report of this Committee. The report on these tests, together with a report on the previous researches, has been published in the National Sand and Gravel Bulletin and as Circular 8 of the National Sand and Gravel Association, and includes data of tests on void content for all possible combinations of three sizes of sand—No. 14 to No. 4, No. 48 to 14, and 0 to No. 48—and for 57 combinations of four sizes of gravel—¾ to 1½-in., ⅜ to ¾-in., No. 4 to ⅜-in., and No. 8 to No. 4.

Effect of Mineral Composition and Shape of Particles in Coarse Aggregate on Strength of Concrete: (National Sand and Gravel Association, Washington, D. C.). This investigation is designed primarily to show what variations in quality of concrete may result from a difference in mineral composition of the coarse aggregate or from a difference in the shape of particle. It includes a large number of different aggregates from widely separated sources tested under comparable conditions. A feature of the tests is the attention paid to differences in the void contents of the different aggregates and the effect of this factor on the quantity of mortar required to produce workability. Both compression and flexural tests are being made.

Effect of a Amall Amount of Colloidal Clay in Sand on Its Usefulness as a Fine Aggregate. (University of Colorado, Boulder). A leading sand and gravel company of Denver has been having trouble in marketing the sand that is obtained in connection with their commercial gravel. The sand and gravel deposit underlies 2 or 3 feet of top soil and is very extensive and uniform. With nominal washing the gravel is entirely satisfactory, but although the sand makes concrete that is satisfactory from the standpoint of strength, contractors have objected that it is "dead" and does not work properly. It has been necessary, therefore, to waste most of the 0-4 material from this pit. After various tests it was found that the presence of a small amount of colloidal clay seemed to be the main source of the trouble. Some of this clay is so fine that water containing it will remain clouded for days. This slime is very tenacious and the adhesion is such as to impart a distinct crustiness to any dried surface of the unwashed sand. When the clay is washed out and dried it resembles hardened cement and has considerable strength in tension. A thorough washing removed all signs of the objectionable "deadness" and no crust formed upon drying. Special cone slime removers and washing equipment were installed as a result of these tests and the sand has apparently been rendered a satisfactory and marketable product. The washing also removed some indications of organic contamination as shown by the colorimetric test.

Effect of Briquet Molding Pressure on Strength Ratio of Sand Mortars: (National Sand and Gravel Association, Washington). This investigation was outlined for the purpose of obtaining information on the effect of different laboratory practices on results of tests of sands in mortar. Preliminary studies indicate that the pressure applied in molding briquets may affect the strength of concrete sand mortars differently from standard sand mortars, and hence may affect the strength-ratio.

Crushing and Abrasion Tests of Gravel: (National Sand and Gravel Association, Washington). In a study of quality of gravel particles, crushing tests and modified Deval abrasion tests of a large number of samples of gravel are being made. The crushing test is being made by applying a pressure of 3000 lb. per sq. in. on a 3 kg. sample of gravel, graded 50 per cent 1 to 1½-in., 25 per cent ¾ to 1-in., and 25 per cent ½ to ¾-in., confined in a cast iron cylinder 6 in. in diameter. The reduction in fineness modulus and in volume of voids due to application of load is being determined. The abrasion test is being made on a sample of the same grading, to which 6 cast iron balls 1⅞-in. in diameter are added as an abrasive charge, revolved 10,000 revolutions in a standard Deval cylinder at 33 r. p. m. These data are included in a progress report to Sub-Committee 2 of Committee 2 on Ballast of the American Railway Engineering Association (Bulletin No. 321, Vol. 31, Nov., 1929, p. 761).

Accelerated Soundness Tests of Gravel: (National Sand and Gravel Association, Washington). Preliminary tests have been carried out to determine whether the difference in abrasion test results of gravel before and after treatment with sodium sulfate, in accordance with the accelerated soundness test, may be used as an indication of the soundness of the material. Results

on gravels studied to date have been unsatisfactory, since a greater resistance to abrasion after treatment with sodium sulfate is shown sometimes than was obtained on the untreated samples.

Durability Studies of Aggregates for Concrete: (Research Laboratory, Portland Cement Association, Chicago). Comprehensive studies are being made of the variations in aggregate quality that may be encountered in pits and quarries and the effect of such variations on the durability of concrete in which the aggregates are used. The study embraces petrographic and geologic studies in the field and laboratory, sodium sulfate tests for soundness of aggregate, freezing and thawing tests of the aggregates and of concretes and mortars in which the aggregates are incorporated. A major objective of this study is the establishment of some physical test by which the suitability of aggregate for use in concrete can be determined.

Well-Graded Washed Sand Versus Crushed Granite Screenings as Fine Aggregate: (University of Colorado, Boulder). For the conditions of these tests the granite screenings gave concretes which were decidedly above those from the washed sand in strength and fully equal to them in workability. These tests were with identical coarse aggregates and water-cement ratios. Mixtures of sand and screenings gave strengths below those for the screenings alone but above those for the washed sand.

Limestone and Gravel Used as Coarse Aggregate: (Pennsylvania Department of Highways, Harrisburg). Tests were made to compare the relationship between strength of concrete in which limestone and gravel were used as coarse aggregate. Modulus of rupture tests were made on concrete beams and compression tests on concrete cylinders at ages of 1 day to 1 year. At all ages the crushed limestone gave higher strengths than the gravel. The modulus of rupture of the limestone concrete at various ages ranged from 10 to 20 per cent higher than that of the gravel concrete. In compression, the limestone concrete was between 20 and 35 per cent higher than the gravel concrete.

Use of Soft Limestone in Concrete: (Iowa State Highway Commission, Ames.) Tests were made to determine the effect on strength and durability of concrete of the use of relatively soft limestone as coarse aggregate. Six by 6 by 30-in. beams and 6 by 12-in. cylinders were made using normal limestone in one group, and a very soft limestone in companion specimens. Strength tests indicate a slight decrease in both flexural and compressive strengths particularly for soft limestone which was found to be unsound in the sodium sulfate soundness test.

Development of a New Strength Test for Sand: (Iowa State Highway Commission, Ames). The purpose of this investigation was twofold: (1) to develop a strength test for sand which will accurately and consistently evaluate the concrete-making ability of different sands, (2) to develop a method of designing concrete mixes on the basis of this test. At various periods for the past 9 years work has been undertaken on these problems using different phases, different approaches, and different ideas. Of all the methods tried, the mortar-

voids method of Talbot and Richart seems to be the most dependable, the nearest approach to the realization of the basic laws which govern the strength of concrete. During the past year, several thousand 2 by 4-in. and 6 by 12-in. cylinders and 6 by 6 by 30 and 2 by 2 by 14-in. beams were made and tested. The method of Talbot and Richart of designing mixes was tried out in the field and used in the construction of about 200 miles of pavement. Tests were made on core specimens from this pavement and on beams made during the construction of the pavement. The technique of Talbot and Richart has been improved upon to such an extent that some of the difficulties they met with have been eliminated, as well as new difficulties which arose during these tests. The technique has now reached the point where the effect of such variables as amount of water required to wet the surface of coarse aggregate variations in manner of mixing and placing, and workability as distinguished from consistency are factors of prime importance. When these factors have been evaluated, it will be possible to design concrete mixes for a definite strength and attain values within a range of considerably less than 10 per cent of the desired result. In addition, it will be possible to estimate accurately quantities of all materials after a few hours' work with the sand and cement to be used in the concrete. The attainment of this is, of course, based on the assumption that the curing condition will be the same, and that the coarse aggregate is structurally sound and of the quality normally used.

Short-Time Strength Tests for Concrete Sands: (Kansas State Agricultural College, Manhattan). This investigation was undertaken in 1925 to determine the relation between the strength-ratios of various natural and commercially available sands in the State, using a high early strength cement and those obtained using portland cement in the ordinary 7 and 28-day strength tests.

The procedure is to test a number of sets of three briquets and three 2 by 4-in. cylinders made with each sand and each high early strength cement at 24 and 48 hours, for comparison with corresponding sets of briquets and cylinders made on the same day from standard Ottawa sand and the same high early strength cements. Similarly 7 and 28-day strength ratios will be obtained for each of the sands with ordinary portland cement. It should be possible to secure certain coefficients for interpreting 24 and 48-hour strength ratios in terms of 7 and 28-day strength ratios. Also, it is possible that curves may be obtained which will show directly the relationship between strength ratios of early strength cements and those of ordinary portland cement with the various sands available in Kansas.

Study of Effect of Character of Fine Aggregate on Flexural Strength of Mortars: (U. S. Bureau of Public Roads, Washington). This investigation has been undertaken to determine the importance of character of fine aggregate on resultant strength of mortar in flexure.

Study of Methods of Determining Absorption and Free Water Content of Fine Aggregates: (U. S. Bureau of Public Roads, Washington). This group of tests is being made to develop, if possible, efficient methods for determining absorption and free water content of fine aggregates.

Study of Methods of Making Abrasion Tests on Fine Aggregates and Hardness Tests of Mortars: (U. S. Bureau of Public Roads, Washington). This is a continuing investigation to study the relation between abrasion tests on fine aggregates and hardness tests of mortars.

Studies of Effect of Type of Aggregate on Resistance of Concrete to Frost Action: (U. S. Bureau of Public Roads, Washington). This investigation has been completed, and a report on the results obtained will be published shortly.

Study of Relation between Coarse Aggregate Content and Quality of Pavement Concrete: (U. S. Bureau of Public Roads, Washington). The object of this investigation was to determine the effect of variations in the amount of coarse aggregate on workability and quality of portland cement concrete in pavement slabs constructed with standard equipment. The study involves the construction of approximately one-half mile of concrete pavement 9 feet wide and 7 inches thick, constructed in 9-ft. sections. Both crushed stone and gravel were used as coarse aggregate and the proportions varied from a distinctly over-sanded mix (1:2:3½) to mixes containing as high as 4¾ parts of crushed stone and 5¼ parts of gravel. Observations were made on the relative workability of the various concretes when finished with both of the standard finishing machines now on the market. The quality of the concrete was determined by flexural tests on concrete beams 27 in. wide and 5 ft. long taken from the test sections. Other tests for quality, such as compression tests on cores drilled from the slabs, and freezing and thawing tests of concrete specimens taken from the beams, were also made.

Investigation on the Quality of Concrete for Bridge Floor Slabs with Special Reference to Use of Light-weight Aggregates: (U. S. Bureau of Public Roads, Washington). This investigation is under way in cooperation with the Port of New York Authority.

3—Plain concrete

The researches under way on plain concrete which are described below have been listed under the following classifications:

Admixtures
Bricklaying
Crazing
Curing
Dams
Durability
Fatigue
Fire Resistance
Flow under Load
Jigging
Mixers and Mixing
Modulus of Elasticity and Poisson's Ratio
Pavements
Permeability
Pipe
Products
Strength—compressive, flexural
Temperature Effects
Tests and Test Methods
Volume Changes
Wear
Workability

ADMIXTURES

Effect of Sugar on Strength of Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). Tests were made on sand, cement, and coarse aggregate which were sent to the Laboratory in sugar containers, such as sacks and syrup buckets. It was found that very small amounts of sugar reduce the strength of mortar to almost nothing, and that very unreliable results are obtained on material shipped in such containers. Fine and coarse aggregate shipped in this manner, however, tested satisfactorily after being thoroughly washed. Results of this investigation were published in Kentucky Highways, v. 3, No. 3, Sept., 1928.

Tests of Admixtures: (California Division of Highways, Sacramento). Tests are under way to determine the effect of additions of various commercial powders on the properties of cement mortars and concretes. Laboratory compression tests are being made on 2 by 4-in. mortar specimens and on cores drilled from pavements. Results not yet available.

Celite as an Admixture: (University of Colorado, Boulder). The strengths of concretes and mortars with 1:2 and 1:3 cement-sand ratios were increased appreciably by additions of Celite of 3, 5 or 10 per cent of the weight of the cement. The mixtures were stiffened by the Celite and at constant workability the required increase in water-cement ratio offset the effect of the Celite. Both fine and coarse aggregates were crushed granite of excellent quality. While the mixtures were not harsh prior to the addition of Celite there was apparent the usual softening or "fine sand" effect. (In some former tests with different cement and aggregates, there was no apparent strength difference between concretes and mortars with and without Celite at constant water-cement ratio). Celite had no measurable effect on yield in either series.

Permeability Tests of Concrete with and without Admixtures of Celite: (Pierce Testing Laboratories, Denver, Colorado, for Johns-Manville Corp.). Permeability tests were made on 1:2:4 concrete mixtures, with and without admixtures of Celite, subjected to water and oil at pressures up to 200 lb. per sq. in. In a 1:2:4 mix with a water-cement ratio of 0.85, the slump for concrete without Celite was 4 in., and that with the admixture in amounts of 3 lb. per bag of cement, the slump was 1.75 in. The compressive strength at 7 days was 2645 and 2910 lb. per sq. in., and at 28 days was 4017 and 4652, for untreated and treated concrete, respectively.

The concretes were cast into 14 by 14-in. concrete cylinders, with a pipe projecting from the center, at the bottom of which was left a cavity 1½ by 1½ in. thus leaving a wall of concrete 6 in. thick in every direction, on which the pressure was applied. These specimens were connected with the apparatus, each having a reservoir equipped with a water or oil gage, thus making it possible to read the amount of water or oil forced into or through each specimen at any time. These reservoirs in turn were connected to an air tank equipped with pressure gage, air compressor, and motor by means of which the pressure was raised to and maintained at the desired amounts. On total

loss of oil, Celite concrete showed a permeability 15.4 with plain concrete considered as 100; on total loss of water, Celite concrete showed a permeability of 29.5 on this same basis.

Effect of Calcium Chloride on Hydration of Minerals: (Ohio State University) This investigation was conducted in the Department of Mineralogy in cooperation with the Calcium Chloride Association. The results obtained will be published by the Engineering Experiment Station.

Effect of Various Powdered Admixtures on Workability of Concrete: (Missouri State Highway Department, Jefferson City). These tests were made to determine the efficacy of various powdered admixtures for improving the workability of concrete. Laboratory tests were made with the Smith-Conahey workability apparatus on concrete containing different admixtures in varying amounts, all batches being gaged with enough water to give a constant flow. The basic mixture used was 1:2:3½ at a flow of 125 (15⅛-in. drops). Two sands were included, one a rounded-grain quartz river sand, and the other an angular-grained crushed flint sand. Field tests and observations were also made of the use of these admixtures in actual construction. Though the investigation has been completed, no report is available for publication.

Effect of Crusher Dust on the Strength and Other Properties of Mortars and Concretes Made from Crushed Granite: (Univ. of Colorado, Boulder). Crusher dust (of the same material as the aggregates 0-100 grading) was added to mortars and concretes as a check on the extent to which accidental concentrations of dust might be harmful to the quality of the concrete. At a constant water-cement ratio the dust strengthened the concrete but stiffened it materially. At constant slump the increase in strength from adding dust was about equal to the decrease from increasing the water-cement ratio up to an amount of dust equal to 15 or 20 per cent of the total fine aggregate. The conclusion from these tests and materials is that no ordinary concentration should be injurious but it may complicate the control of mixing water and workability. The results from these tests should not be extended to other conditions and materials without experimental verification, however.

Use of Diatomaceous Earth in Concrete. (Pennsylvania Department of Highways, Harrisburg). This investigation was undertaken to determine the effect of admixtures of diatomaceous earth on the properties of concrete. Concrete cylinders and beams were molded using the same water-cement ratio and same materials, except that diatomaceous earth was added to one group of specimens in amount of 3 lb. per bag of cement. Concrete containing the admixture was found to be more plastic, and the surface of the specimens smoother, while the plain concrete had a higher modulus of rupture and higher compressive strength. The admixture did not reduce the absorption of the concrete.

Use of Calcium Chloride in Concrete: (Iowa State Highway Commission, Ames). The purpose of this investigation was to determine the effect of calcium chloride used as an admixture in concrete. Two series of 6 by 6 by

30-in. beam specimens were prepared, one set during the construction of a paving project which was started in 1926, and the other set in the laboratory. For the field series the principal variables were curing condition and type of aggregate. For the laboratory specimens a number of variables was introduced, such as different brands of cement, various types of coarse aggregates, and different consistencies. A final report will be prepared supplementing the progress reports of January and August, 1927.

Use of Admixtures in Concrete: (Iowa State Highway Commission, Ames). These tests were undertaken to determine the effect of various inert admixtures in concrete, including pumicite, hydrated lime, Celite, Morrisite, and portland cement. Specimens tested were 6 by 6 by 30-in. beams and 6 by 12-in. concrete cylinders containing varying amounts of the admixtures. The strength tests obtained to date indicate little or no advantage in using admixtures in the amounts recommended by the producers. Some of them, particularly Celite, increase the yield slightly, and somewhat out of proportion to the absolute volume of the material added. This appears to be due to the additional water required. Workability, the talking point of the producers, is as yet unmeasured by any means which affords a common measure for concrete made with different admixtures.

Tests of Cements and Admixtures: (Delaware, Lackawanna, and Western Railway Co., Hoboken, N. J.). 6 by 12-in. cylinders were made for test at 7, 28, 60, 180, and 360 days from concrete with and without 5 per cent admixtures by weight of cement of Celite, Cal, Dowflake, Colloy and Fluresit, using two standard portland cements as well as Lumnite and trass cements. Two water-cement ratios were used for each group—6.25 and 6.75 gal. per sack—the aggregates were dry, and the slump was 3 in. in all tests. In tests planned for future work Incor cement will be included to determine its early strength quality and its strength up to one year.

BRICKLAYING

Investigation on Bricklaying Problems: (Mellon Institute, University of Pittsburgh, Pittsburgh). This series of tests was undertaken by Mellon Institute, in collaboration with the Eastern Face Brick Manufacturers' Association, to study those combinations of variables representing the probable range of practical application. About 400 experimental walls and panels were erected in the grounds of Mellon Institute Building. Since the most obvious point of attack are studies of the characteristics of different kinds of brick and mortar, the brick and mortar problems under investigation include the absorption and surface characteristics of brick, and differences in mortars due to varying the cementing materials, sand, and pigments. Properties of mortars being studied are workability, compressive and transverse strength, absorption and permeability, shrinkage, durability, staining and efflorescence, and elasticity. Information is also being collected to determine whether the 10 per cent lime often specified by architects, or the 25 per cent or more usually employed by brick masons, gives better results.

The problems in backing include a study of hollow tile of different sizes, of brick, of cinder or concrete block, of brick tile, and of metal or other lath on

steel frame. Variations in workmanship are most apparent in regard to tapping, pointing, and the filling of head-joints. Tapping is being studied not only with relation to the effect of excessive downward tapping into place, but also with relation to plumbing both before and after initial set.

Special mention should be made of weather problems. The experiments are planned for the purpose of investigating the behavior of mortar with reference to the other variables in all types of climatic conditions. Summarized recommendations as well as the complete report are being published by the American Face Brick Association.

CRAZING

Development of Hair Cracks into Structural Cracks: (University of Michigan, Ann Arbor). This investigation was started in 1926 to determine whether hair cracks develop into structural cracks on alternate wetting and drying of neat cements and rich mortars. Though the data are being obtained very slowly, it appears that hair cracks become larger and deeper with alternations of volume change.

CURING

Rates of Evaporation and Absorption of Mortar and Concrete Specimens of Different Sizes and Shapes: (University of Colorado, Boulder, in cooperation with the U. S. Bureau of Reclamation and the Arch Dam Committee of Engineering Foundation). By frequently weighing large numbers of specimens of different sizes and shapes as they were exposed to the air after different periods of curing in water a very large quantity of data on the rates of moisture loss have been collected. Among the sizes and shapes of specimens were cylinders (2 by 4-in., 3 by 6-in., 3 by 12-in., 3 by 18-in., 4 by 14-in., and 6 by 12-in.) and beams (3 by 3 by 40-in., and 4 by 6 by 38-in.). It is reasonable to expect the rate of moisture loss to be more or less proportional to the surface volume ratio for a given mixture, period of moist curing, and type of exposure. This presumption was borne out in part. Other tests were made to ascertain the absorption rates upon re-immersion. The absorption rate is much higher than the rate of moisture loss, very substantial quantities of water being taken up by dry concrete during the first few seconds of immersion. These data are in process of preparation for publication.

Curing Concrete Pavements: (Kentucky State Highway Department and University of Kentucky, Lexington). The purpose of this investigation was to determine whether pavement protected with wet burlap for 24 hours is as satisfactory as that covered on the surface with calcium chloride following the 24-hour wet burlap curing. The concrete used was a 1:2:3.5 mix by volume field condition, with an average cement factor of 1.52. Washed bank sand and gravel of satisfactory gradation were used with standard portland cement averaging about 300 lb. in tensile strength at 7 days. Two 7 by 7 by 30-in. beams were made each day for 20 days; both were covered with wet burlap for the first 24 hours, after which one beam of each pair was surface-treated with calcium chloride. The specimens were tested in flexure and compression

at 7, 14, 21 and 28 days. During the construction of the pavement the concrete was covered over with wet burlap for the first 24 hours, after which every alternate 50-ft. section was treated with calcium chloride. Two cores were taken from each treated and untreated section. The results obtained from these tests indicate that there is no advantage in using calcium chloride, since almost equal strengths were obtained for both treated and untreated concrete, and the untreated concrete developed sufficient strength at 7 days both in flexure and compression to permit opening the pavement. A study of cracking of the pavement did not show any advantage of the treated over the untreated concrete.

When the project was nearing completion it was found that the supply of calcium chloride was exhausted. It was decided to omit calcium chloride from the last 3000 ft. of pavement, but to treat 2000 ft. of concrete with silicate of soda. The small amount of work done in this connection indicated that there is nothing to be gained by using this material.

Imperial Valley Curing Investigation: (California Division of Highways, Sacramento.) An extensive investigation is under way to determine the efficacy of various methods of curing concrete pavements. Bending tests are being made on beams cast in pavement and on cores drilled from the concrete. Tests made up to and including 28 days on all curing methods showed lower strengths than standard water curing. At 3 months, three of the methods gave slightly higher strengths.

Effect of Methods of Curing on Strength and Wear Resistance of Concrete: (Texas A. & M. College, College Station). Tests are being made to determine the strength and resistance to wear of 250 beams and 500 cylinders cured by means of calcium chloride, Hunt process, curcrete, silicate of soda, wet earth, regulated sprinkling, and no curing.

Curing of Concrete: (Ohio State University, Columbus). Researches were made to determine the effect of calcium chloride as a curing agent for portland cement concrete. These studies were made as a part of a graduate thesis, in cooperation with the Calcium Chloride Association, and will be published by the Engineering Experiment Station.

Comparison of Methods of Curing Concrete Pavements: (Wisconsin Highway Commission, Madison). This investigation covered comparative tests of various methods of curing, using wet earth curing as the criterion. The tests were run in 3 series of five 400-ft. curing sections. One-half the slab was cured with wet earth, and a wood header was placed down the center joint and tarred thereto to prevent seepage. The other half was treated with the various methods included in these studies. Three sets of beams of two each, one earth-cured and the other with special curing, were cast within road forms, and hand-finished composite specimens were made outside road forms. In addition, cores were taken at various ages.

Loss of Moisture from Concrete Cured by Various Methods: (Missouri State Highway Department, Jefferson City). The purpose of these tests was to

determine the relative efficiency of various curing methods in keeping concrete moist during the curing period. Weighed samples of concrete were placed in enameled pans 12 in. in diameter and 4 in. deep. Two samples were cured by each of the following methods:

(1) Wet burlap for eighteen hours followed by
    (a) Calcium chloride
    (b) Sodium silicate
(2) Curcrete applied immediately after molding.
(3) Hunt Process applied immediately after molding.
(4) Calcium chloride used integrally.

Though these tests have been completed, no report has yet been prepared for publication.

Effect of Different Methods of Curing on Strength of Concrete: (Pennsylvania Department of Highways, Harrisburg). This investigation covered a comparison of the value of wet straw, calcium chloride used integrally, and asphaltic emulsion sprayed on the surface as curing agents. Sections of a state highway were cured by these methods. Specimens were molded for strength determinations at various ages, and cores were taken from the road at the six-month and one-year periods. The strength of beams, cylinders, and cores was about equal. At the six-month period the calicum chloride and straw-cured cores gave a strength of 4100 lb. while those cured with the asphaltic emulsion were slightly lower. The beams and cylinders cured with calcium chloride were slightly higher at 7 and 10 days than either of the others, but at the 14 and 28-day periods the strengths were about equal.

Methods of Curing Concrete: (Iowa State Highway Commission, Ames). This problem was concerned particularly with curing of concrete pavement slabs. The investigation was conducted in four groups, each of which consisted of tests on slabs 10 ft. by 10 ft. by 6 in., made in a manner approximating ideal field conditions, and including from four to fifteen methods of curing. In the construction of the slabs longitudinal divider boards were spaced two and one-half feet apart, thus dividing the slabs into four strips 2½ ft. wide and 11 ft. long. These strips were removed one day before testing at ages of 7, 28, 90 and 360 days. Each strip was sawed transversely into 6 by 6 by 30-in. beams on which flexural tests were made. Compression tests were made on specimens prepared from the beams. The results indicate that the wet burlap moist earth method of curing is the most efficient method available; that other methods afford strength results of varying percentages of the results obtained for this method; that asphaltic coatings such as Curcrete and Hunt Process are 90 and 95 per cent as effective as this method; that calcium chloride on the surface is about the same; and that the original water content of the concrete must be maintained or even augmented for the most efficient curing.

A laboratory study has been undertaken to investigate the latter point, and to determine the effect of different methods of curing on cracking, scaling, and

volume changes of the concrete. The work on volume changes is well under way and the data obtained indicate greater daily volume changes for asphaltic coatings than for the other methods, especially the standard earth and water method.

Comparative Study of Wet Earth, Calcium Chloride, and Sodium Silicate for Curing Concrete Pavements: (U. S. Bureau of Public Roads, Washington, in cooperation with the Maryland State Roads Commission). A description of this project as well as the results of tests made up to one year was published in the September, 1928, issue of Public Roads. Observation of service behavior is made from time to time.

DAMS

Temperature Studies in an Arch Dam: (University of Alabama, University, Ala.). The purpose of this investigation was to determine the maximum temperature occurring in concrete after placing, the temperature at time of closure of the dam, and the annual range in temperature. Thermocouples were set in the concrete at time of construction, and readings have been taken each month for the past year. This investigation will probably be continued for another year.

Investigation of Models of Arch Dams: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Engineering Foundation). This extensive study was undertaken to add to existing knowledge of stress distribution for design purposes, and is a part of the preliminary investigation of the Boulder Canyon Dam. The report of this Committee for 1928 (see page 444, Proceedings, A. C. I.), carried a complete outline of tests to be made on a model of the Stevenson Creek Dam, which have since been completed. Experimental work of similar nature was carried out on a model of the Gibson Dam, with auxiliary tests in flexure, tension, torsion, compression, volume changes, etc., somewhat less elaborate than those carried out on the Stevenson Creek model. It appears from the results obtained to date that models have great possibilities as aids to structural analysis of dams, as have also the auxiliary findings.

Model Tests on Hoover Dam: (U. S. Bureau of Reclamation at the University of Colorado, Boulder, in conjunction with Arch Dam Committee of Engineering Foundation). A model of plaster of Paris diluted with Celite to reduce the modulus of elasticity sufficiently to give measurable deformations is under construction. Young's modulus will be between 100,000 and 300,000 lb. per sq. in. and the model will be to a scale of about 1:200. Very extensive tests on cylinders (2 by 4-in., 3 by 6-in., and 6 by 12-in., on 3 by 3 by 40-in. and 4 by 6 by 38-in. beams) have been made on a great variety of mixtures and materials in finally selecting the materials and proportions to be used in the model. Extensive tests on the rate of drying out have been made, since plaster of Paris is plastic when moist and must therefore be in a dry condition at test. The methods of test will be similar to those used for the Stevenson Creek and Gibson models.

DURABILITY

Durability of Concrete: (Hydro-Electric Power Commission, Toronto, Canada). During the past year most of the work done in this investigation was a field study of the behavior of concrete structures under different conditions of service, with a total inspection of about seventy-five structures. In addition, laboratory studies are being made of different natural waters to determine their corrosive effect on concrete. The monthly variation in composition is being obtained of fifteen different waters ranging in type from a very pure, soft water, originating from swamps to a relatively hard water. The products resulting from the seepage of these various waters through the concrete are being determined, and an attempt is being made to duplicate in the laboratory the action of corrosive water on concrete as observed in the field.

A 3-yr. study of the efficacy of colorless waterproofing treatments for concrete surfaces, which forms a part of this investigation, has been completed. The results indicate that few of these treatments have any permanent waterproofing or protective value when applied to concrete surfaces.

Methods of Rendering Concrete Drain Tile Resistant to Alkali Attack: (Iowa State College, Ames). This project covers long-time observations of concrete samples which were treated with various chemicals and inert materials before immersion in alkali solutions. A report of this investigation will be published during 1930.

Resistance of Concrete to Waters of Acid Reaction: (State of Washington Department of Highways, Olympia). The purpose of this extensive investigation, which was started in 1927 and which will probably not be completed before 1931, is to determine the nature and extent of attack by waters having an acid reaction, and to study means of making concrete resistant to such attack. One series of tests has already been completed, and another is under way on tests of specimens exposed to artificially acidified slowly running water in the laboratory, and to natural waters of acid reaction. It is intended that the second series will cover such points as (1) relationship between volume of concrete and surface exposed, (2) depth of attack from the surface, (3) protection afforded by coatings and admixtures, and (4) comparison between artificially acidified water in the laboratory and a naturally acid stream.

Durability of Concrete: (Kansas State Agricultural College, Manhattan). The purpose of this investigation was to determine the resistance of concrete and concrete aggregates to alternate freezing and thawing and to exposure to alkali water, as well as to determine the effect of porosity on durability. Freezing and thawing tests were made on 300 concrete specimens of various water-cement ratios and mixes, and on 200 aggregate samples.

Field specimens were placed in 4 locations in Kansas streams known to contain alkali. The following are some of the results obtained:

(1) Field specimens of 1:5 or leaner mix show continued disintegration. Specimens of mixes richer than 1:5 are unaffected.

(2) From tests of specimens exposed to freezing and thawing it appears that:

(a) Alternate freezing of saturated concrete at low temperatures and thawing at room temperatures are a valuable means of determining durability of concrete.

(b) Variations in water-cement ratio of plastic, workable concrete have a very marked effect on its durability. A water-cement ratio of 1.0 is near the maximum permissible to obtain desirable durability of concrete under conditions of exposure.

(c) Use of unsound aggregate results in unsound concrete the resistance of the mortar to distintegration being only slightly effective in protecting the aggregate.

Deterioration of Concrete Silos: (Kansas State Agricultural College, Manhattan). The purpose of this investigation was to determine the causes of deterioration of portland cement concrete when exposed to action of ensilage liquors, and to develop methods of overcoming and preventing such deterioration. The tests were undertaken as the direct result of numerous requests received during the past five years for information on means of protecting the interior surfaces of silos from deterioration by the ensilage juices.

In carrying out this investigation, portland cement concrete specimens were prepared with varying mixtures and varying types of surface treatments, and tested for permeability before and after exposure to the silage juices. Synthetic solutions of ensilage liquors were prepared and the specimens were exposed to the action of these solutions under varying conditions of submergence, air exposure, temperature and concentration of solution. The interiors of five silos in the vicinity of Manhattan were treated with ten different protective coatings. The interior of each silo was divided into equal areas (usually 6 or 8), extending the full length of the silo, each area being treated with a different material. These treated silos are being examined periodically to determine the effectiveness of the surface coatings.

Protective Treatments for Concrete against the Action of Salt Waters: (U. S. Bureau of Public Roads, Washington). Mortar specimens treated in various ways are installed in the alkali water of Medicine Lake, South Dakota, and their condition noted from time to time.

FATIGUE

Fatigue Tests on Concrete: (Iowa State Highway Commission, Ames). Studies are under way to determine the effect of repetition of stresses on concrete. Tests were made on 6 by 12-in. and 4½ by 9-in. cylinders to determine effect of repeated loadings on modulus of elasticity and strength in compression. Some work was also done on the number of repetitions at different percentages of the ultimate loading. Apparently a relatively small number of repetitions of 62.5 and 75.0 per cent of the ultimate stress will cause failure.

FIRE RESISTANCE

Fire Resistance of Walls of Concrete Masonry Units: (Research Laboratory of the Portland Cement Association, Chicago). In this investigation the

various factors bearing on the performance of walls of concrete masonry units when subjected to standard fire exposure are being studied. The investigation was started in June, 1928 with the design and construction of suitable equipment for the conduct of standard fire tests and the installation of a block machine and mixer for the manufacture of the units. To date fire tests have been conducted on more than 80 wall panels 5½ by 6 ft. in size representing approximately two thirds of the test work scheduled for the investigation.

The test program is arranged to determine the influence of such factors as grading of aggregate, cement content, character of aggregate, design of masonry units, type of mortar, workmanship in laying up the walls, and application of plaster. In general, the tests are scheduled in groups or series to bring out the effects of each factor over a wide range. As the work progressed it was found desirable to include several miscellaneous tests. Valuable information is being obtained regarding the stability and safety of walls of concrete masonry both before and after severe fire exposure.

JIGGING

Effect of Jigging during Setting Period on Compressive Strength of Concrete: (University of California, Berkeley, Calif.). Tests were made on 1:4 concrete having a ¼-in. slump. For one group, the aggregate had an appreciable excess of fines, and in another group the proportion of fine aggregate was the minimum which would produce a workable mix. After molding, the specimens were subjected to jigging, which varied from one-half to 1½ hours and from 33 to 154 vibrations per minute, in accordance with methods described in the 1927 report of this Committee.

The maximum increase in strength of the jigged over corresponding unjigged specimens was 40 per cent. The average increase for Group 1 was 13 per cent and for Group 2, 19 per cent. The highest rate of jigging gave the best results for each group. For this condition, the average increase in strength for Group 1 was 31 per cent, and for Group 2, 23 per cent.

A second series of tests was conducted using two water-cement ratios, two gradations of aggregates, and two diatomaceous earths in varying proportions. The compressive strength was increased by jigging for all mixes used, and increased with time of jigging up to the maximum of one hour used in this series. Concrete made from an aggregate with an excess of fine material was benefited less than that made from a properly proportioned aggregate.

FLOW UNDER LOAD

Flow of Concrete under Continuous Compressive Stress: (University of California, Berkeley.) This investigation is being conducted in 3 series, the first of which includes tests on 4 by 14-in. cylinders subjected to constant compressive stress at 70 degrees F. as previously described in the 1928 report of this Committee.

The second series of flow tests has been started to determine the effect of type of aggregate and the effect of moisture conditions upon flow. All specimens were loaded at 28 days to 800 lb. per sq. in. The net flows (after cor-

recting for the changes in unloaded control specimens) at age of 90 days were as follows:

Type of Aggregate	*Flow in Millionths of Inches per Inch*
Sandstone	710
Basalt	680
Gravel	565
Granite	465
Quartz	387
Limestone	313

The third series of tests to determine the relative longitudinal and lateral flow of plain and reinforced concrete cylinders is now in progress. The cylinders are 10 in. in diameter and 20 in. long, two of them being reinforced with six ½-in. square bars enclosed within an 8-in. spiral. One reinforced cylinder and 1 plain cylinder were loaded to produce a stress of 800 lb. per sq. in. in the concrete. Test observations up to the age of 90 days show that the deformation due to load in the reinforced specimen is approximately ⅔ the deformation of the plain specimen at all ages. For the unloaded specimens the shrinkage deformation of the reinforced specimen is approximately ¾ that of the plain.

Flow of Concrete under Continuous Compressive, Flexural, and Tensile Stress: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Arch Dam Committee of Engineering Foundation. See Proc. A. C. I., 1929, p. 444 for outline). Data have been obtained for loadings at various ages under air and water curing and the results are being worked up for publication. In general the findings accord with the published results of Davis and others in so far as the tests are parallel.

Plastic Flow of Concrete: (Ohio State University, Columbus). A correlation is contemplated of all work carried out on this subject to date for the purpose of publishing it, together with methods of application to engineering design computations.

MIXERS AND MIXING

Investigation of Concrete Paver Operation: (Minnesota Highway Department, St. Paul). The purpose of this experiment was to check the water supply equipment in use and the amount of concrete remaining in mixer drums from one batch to the next, and to make a time study of the mixer cycle on Minnesota paving jobs. Stop watch studies of the mixer cycle, water measuring tanks were calibrated, and the water supply equipment was inspected. The latter was usually found to be adequate, though the valves on water measuring tanks were inefficient in several cases. There was considerable variation in different mixers in time of charging, discharging, and entrance of water into the drum, which resulted in a variation in the time interval necessary for mixing and for complete cycle.

MODULUS OF ELASTICITY AND POISSON'S RATIO

Effect of Moisture on Strength and Modulus of Elasticity of Concrete: (University of California, Berkeley). Tests were made at ages of 35 and 142 days

on 6 and 12-in. cylinders of three water-cement ratios. Half the specimens were left in water until the time of test, and the remainder removed from the water and stored in dry air at 130 degrees F. for 7 days prior to test. All were tested at 70 degrees F. The results are given below:

Mix	*Strength, lb. per sq. in.*				*Secant Modulus at 800 lb. per sq. in.*			
	35 Days		*142 Days*		*35 Days*		*142 Days*	
	Dry	*Wet*	*Dry*	*Wet*	*Dry*	*Wet*	*Dry*	*Wet*
1:1⅔:3	3740	3150	5940	5400	2,770,000	3,270,000	4,060,000	4,640,000
1:2½:3	3120	2470	4870	3840	2,510,000	2,940,000	2,960,000	3,520,000
1:3¾:4½	1590	1400	2520	1695	2,020,000	2,740,000	2,100,000	2,800,000

It is evident that the effect of moisture is to lower the strength and to increase the modulus of elasticity, the strength range being relatively greater for rich mixes than for lean ones, and the modulus of elasticity range being relatively greater for lean mixes than for rich ones.

Modulus of Elasticity of Concrete (State College of Washington, Pullman). Data have been collected from the results of routine testing of concrete cylinders in the field as well as in the laboratory to determine whether concrete can be considered elastic, and if so, to determine the relation between the proportional limit and ultimate strength, as well as to determine the modulus of elasticity. The results of these tests were published in Bulletin 30 of the Engineering Experiment Station, in April, 1930.

Modulus of Elasticity and Poisson's Ratio of Concrete: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Arch Dam Committee of Engineering Foundation). In connection with all compression tests now made in this laboratory both lateral and longitudinal strains are measured continuously to failure. A semi-autographic apparatus is used in connection with dial extensometers and compressometers. Hundreds of tests have been conducted in this manner and quantities of data have been obtained from 2 by 4-in., 3 by 6-in. and 6 by 12-in. compressive specimens for many different mixtures, aggregates, ages, and curing conditions. Some of these data are in preparation for publication.

Modulus of Elasticity and Poisson's Ratio (National Bureau of Standards, Washington). Tests have just been started on 24 by 48-in. cylinders made at the site of the Calderwood Dam in Tennessee from concrete containing crushed stone fragments ranging in size from five to six inches in diameter, to determine the deformations under load. Longitudinal deformations are being measured during the loading over 32-in. gage lengths along 6 lines equally spaced around the periphery, and also in 2-in. gage lengths at five different elevations. In addition, transverse deformations are being measured in five diameters at different elevations. It is hoped that the data obtained will be sufficient for a satisfactory determination of the modulus of elasticity and Poisson's ratio of the concrete.

Elastic Properties of Concrete: (Kansas State Agricultural College, Manhattan). This investigation has just been undertaken to determine the

modulus of elasticity and Poisson's ratio for concrete, particularly for that made from sand and gravel. At present the extent of the investigation is the trial of several different types of measuring instruments and the development of a new type.

Modulus of Elasticity in Tension, Torsion, and Flexure: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Arch Dam Committee of Engineering Foundation). Many tests have been made on 3 by 12-in. tensile and torsion specimens, and on 3 by 3 by 40-in., and 4 by 6 by 38-in. beams for different ages and curings for the mixtures of the Stevenson Creek and Gibson models. Loads and elongations were taken for tensile specimens; loads and detrusions for torsional specimens; and loads and deflections for flexural specimens. The increment method was used since semi-autographic facilities for continuous testing are not available for these tests. Results are in preparation for publication.

PAVEMENTS

Stresses in Concrete Pavement Slabs under Concentrated Static Loads: (Iowa State College, Ames). This project was started in the spring of 1929 and complete data have been assembled on tests of one small slab, a report on which will be available at an early date.

Correlation of Pavement Condition and Soil Survey: (Michigan State Highway Department, Lansing). Tests were started in 1925 to determine and evaluate those factors which influence the condition of pavements. Three thousand miles of pavement have been studied, including field sketches of pavement condition summarized slab by slab, and correlation of the condition with slab characteristics, such as design, age, traffic, material, curing, reinforcement, character of subgrade, old road beds, and drainage. No definite conclusions are available at the present time.

Study of Methods Commonly Used for Removing Ice from Concrete Pavement Surfaces: Michigan State Highway Department, Lansing). This investigation is being made to determine the most economical and efficient material for removing ice with no detrimental effect on the concrete. Laboratory beams were made, frozen a number of times, and the surface film of ice removed with an application of sodium chloride, calcium chloride, or hot sand. Subsequent observations will be made of the surface condition when exposed to drying atmosphere.

High Early Strength Concrete in Pavements: (Pennsylvania Department of Highways, Harrisburg). A service test road was constructed to determine the relative efficiency of high early strength concrete obtained by (a) high early strength cement, and (b) a special mix. Sections of the road were built using high early strength cement in a 1:2:3¾ mix, standard portland cement in the same proportions and standard portland cement in a 1:1½:2½ mix. The project included four traffic lanes, two of which contained high early strength cement, and the other two the standard portland cement used in the 1:2:3¾ and 1:1½:2½ mixes. After approximately one year of traffic the road sections are all in excellent condition. The special mix and the mix using high

early strength cement were opened at one and two-day periods, depending on the beam strength of the concrete.

Effect of Variable Subgrade Conditions, Various Construction Practices, and the Use of Joints of Various Types on the Service Behavior of Concrete Roads: (U. S. Bureau of Public Roads, Washington, in cooperation with the Virginia State Highway Commission). A description of the various test sections of the Virginia Demonstration Road was published in the June, 1928, issue of Public Roads. Observations of the behavior test sections are being made from time to time.

PERMEABILITY

Permeability Tests of Concrete: (Kansas State Agricultural College, Manhattan). Permeability tests were made on 440 mortar specimens of various mixes and water-cement ratios at pressures varying from 10 to 100 lb. per sq. in. In these tests 3 by ¾ in. cylinders were cured in the moist closet for 24 hours and stored until test in stone jars containing distilled water. The water coming through the specimens was timed for rate of flow, and then titrated with N/20 HCl, using phenophthalein and methyl orange as indicators to determine the amount of lime which was dissolved. In another group of tests 1½-in. discs were sawed from 6 by 24-in. concrete cylinders made from portland cement and Quikard cement in the following mixes: Cement, sand from the Kansas River with a fineness modulus of 2.94, and crushed sandstone in the proportions of 1:1¾:3½, cement and total aggregate in the proportions of 1:3½, the water-cement ratio in each case being 0.7.

The results thus far show that a 1:2 mix was watertight at any pressure up to and including 100 lb. per sq. in. if enough water was used to make a good workable mix, but not so much that the mix is sloppy. That is, a water-ratio of 0.6 to 1.0 may be used, but 0.65 to 0.8 would be better. A 1:3 mix was watertight between the water-ratios of 0.6 and 1.0 and a 1:4 mix between the water-ratios of 0.7 and 1.0 at all pressures up to 100 lb. per sq. in. A 1:5 mix with a water-cement ratio of 0.8 was watertight up to 60 lb. pressure, but water flowed through when more pressure was applied, or if a higher or lower water-cement ratio was used.

Permeability of Concrete: (University of California, Berkeley). An investigation has been outlined to determine the effect of water-cement ratio on permeability of 2 by 6-in. discs.

Permeability Studies: (Portland Cement Association Research Laboratory, Chicago). Studies were inaugurated in the spring of 1928 to determine the influence of various factors on the watertightness of concrete. Apparatus was developed which permitted the making of simultaneous tests on a large number of specimens. The apparatus is so constructed as to permit measurement of the water passing through the specimens which consist of 6-in. diameter discs of varying thickness. The specimens may either be cast in special molds or cut from standard 6 by 12-in. cylinders.

Thus far tests have been made to determine the effect of the following factors on the permeability of concrete: Quality and quantity of cement paste, quantity and type of cement, duration of moist and air-curing, age at test, duration of the test period, thickness of specimen, amount of pressure applied, type of unit, method of molding, direction of application of pressure, uniformity of results, and influence of powdered and liquid admixtures. Results of some of the early tests are reported in the December, 1929, JOURNAL of the Institute.

Oilproofing Concrete (Detroit Edison Co., Detroit, Michigan). The purpose of this investigation was to develop a concrete, or a surface treatment for concrete, which would prevent the absorption of oil from transformers. Seven commercial surface coatings, 3 special cements, 2 integral compounds, and 10 concrete toppings of various mixes, aggregates, and methods of application were investigated. The concrete was molded into saucers 22 by 22 by 6-in. with a 2-in. depression in the top to hold the oil. These specimens were cured for 7 days, dried out for 14 days, treated with various coatings, and filled with light oil of the type used in transformers, the oil being left in the depression for various periods between 9 and 28 days. The specimens were then broken and a careful study made of the cross sections. The results obtained show that concrete can be made or surface treated in such a manner that it will shed oil without any absorption.

Waterproofing Concrete: (Iowa State Highway Commission, Ames). In this investigation tests were made to determine the efficacy of various methods of waterproofing concrete by surface treatments. A number of specimens were coated or painted and immersed in water, after which they were removed and weighed at fixed intervals. From the results obtained it appears that several relatively cheap asphaltic coatings would be quite satisfactory as waterproofers for concrete.

PIPE

Study of Loads on Concrete Pipe in Embankments: (Iowa State College, Ames). This study was continued with the collection of special data on the correlation of laboratory strength of pipe with field strength, and additional studies were made to determine the settlement, changes in diameter, and other phenomena necessary to correct interpretation of the loads caused by embankments.

Strength of Concrete-Encased Clay Sewer Pipe: (Iowa State College, Ames). The results of this investigation were published in the autumn of 1929 in Bulletin 93 of the Iowa Engineering Experiment Station. In addition to the results of strength tests of encased clay pipe, the bulletin includes data on the effect of temperature on strength of concrete.

PRODUCTS

Physical Properties of Cast Stone: (National Bureau of Standards, Washington). This study was undertaken with a view to establishing a federal specification for cast stone. Compression tests were made on 2 by 2-in. cylinders and the modulus of rupture was determined on 1 by 1-in. prisms using a 6-in. span.

In addition, rate of absorption tests were made, and weathering effect was determined by freezing at a temperature of 10 degrees F.

STRENGTH, COMPRESSIVE

Strength of Cement Mortar and Concrete: (Iowa State Highway Commission, Ames). The purpose of this study was to determine the relationship between the strength of cement mortar and the strength of concrete. Particular attention was given to comparison of values obtained in tensile tests of standard mortar briquets and compression tests of field specimens made during the construction of paving projects. The mortar strength tests were made as a part of the routine tests conducted for the acceptance of cement shipped to each paving project. Correlation of these tests with strength tests on field specimens was found to be practically impossible because of the number of variables encountered and the complexity of their relationships. Though work has been temporarily discontinued, these studies will be resumed when the effect of the variables can be more accurately evaluated.

Effect of Adding One-Half to Three-Quarters Inch Gravel and Also One-Half to Three-Quarters Crushed Quartzite Particles to a Neat Cement Paste: (University of Colorado, Boulder). These were miniature "plum or cobble" concretes (specimens about 3¼ inches in diameter). They were cast in watertight molds and only such an amount of stone as could be accommodated easily was added. The "cobble" specimens had compressive strengths about half those of the same paste without any "cobbles." The water-cement ratio was 42 per cent by weight. The weakening was probably due in part to lack of homogeneity (stone bearing on stone, splitting and wedging action) but may also have been traceable to bond failures between stone particles and paste.

STRENGTH, FLEXURAL

Relationship between Transverse and Compressive Strength of Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). A compilation was made of all available results obtained since 1925 in an effort to determine whether transverse or compressive strength results should be considered the fairer criterion of the quality of a concrete. Transverse tests were made using third point loading, and compression tests were made either on 6 by 12-in. cylinders or on pieces cut from broken beams. Below is a brief tabulation of the more important results:

Age in Days	*Strength, lb. per sq. in.*		*Factor*	*Number of Tests*
	Transverse	*Compressive*		
28	1010	3833	3.82	12
21	934	3737	3.99	12
14	936	3596	3.83	12
7	752	2833	3.76	4
28	616	3761	6.08	30

In addition, 23 tests at 28 days showed from 440 to 1141 lb. transverse strength and from 2250 to 5386 lb. in compression with a factor of from 3.84

to 7.03. It is evident that the aggregate is a factor affecting the transverse and compressive strength of concrete, and the results indicate that the compression test is the more likely to reflect the true value of the material. It seems feasible to state that concrete giving satisfactory compressive strength will also give satisfactory strength in flexure, and the resultant concrete will function properly in a pavement.

Comparison of Compressive Strength and Modulus of Rupture: (University of California, Berkeley). For the mixes tested, giving compressive strengths of 2000 to 4000 lb. per sq. in. at 28 days, the modulus of rupture was found to be approximately

$$(S_C - 2000)\ .11 + 450$$

where S_C is the compressive strength of the concrete.
The actual modulus may vary from the computed value by as much as 90 lb. per sq. in.

Masonry Flexure Tests: (University of California, Berkeley). Tests are in progress to determine the flexural strength of brick and tile masonry beams using a variety of cement-lime mortars, the beams varying in size from 8 by 4 by 24-in. to 24 by 12 by 54-in. Nearly 500 specimens are to be tested, including a group of adhesion and bond specimens.

Flexural Tests of Plain Concrete Beams: (New Hampshire University, Durham). This study was undertaken to determine the relation in modulus of rupture between the standard Olsen Laboratory Machine and certain field testing machines, thereby determining the exact stress in beams broken in the field. One hundred sixteen beams were made from water-cement ratio concrete designed for strengths of 2000, 2500, 3000, 4000, and 5000 lb. per sq. in. Such variable factors as curing, time of mixing, size of aggregate, etc., were carefully controlled to insure uniformity in concrete. Companion specimens for each batch were tested on each type of machine, the beams tested in laboratory on the Olsen machine being loaded at the third points of the span.

Standardization of Method for Determining Flexural Strength of Plain Concrete: (American Association of State Highway Officials). The purpose of this investigation was to design and adopt a standard apparatus and method of molding specimens for the determination of the flexural strength of plain concrete. Special apparatus has been designed and built, which is to be used in conjunction with a power testing machine. It is so designed that all forces applied to the specimen are coplanar and perpendicular to its axis, and the deformation due to applied load is entirely unrestrained.

Standardization of Methods of Making Flexural Tests of Concrete: (U. S. Bureau of Public Roads, Washington). These tests are being made in cooperation with A. S. T. M. Committee C-9 on "Concrete and Concrete Aggregates."

TEMPERATURE EFFECTS

Effect of Temperature at Time of Test on Strength of Concrete: (University of California, Berkeley). Tests were conducted on 6 by 12-in. concrete cylin-

ders of 1:2:4 and 1:3:6 mixes at ages of 28 and 135 days. All specimens were wet-cured, but a few days prior to test various groups were subjected to temperatures ranging from 25 degrees F. to 130 degrees F.; some were dried while others were kept wet. In general, the test results show a progressive decrease in strength with an increase in temperature at time of test. The maximum effect developed in the 1:2:4 wet specimens tested at 135 days, the strength ranging from 140 per cent to 85 per cent of the strength at 70 degrees F. The leaner mixes were not affected so markedly as the 1:2:4 mixes, nor the dry specimens so much as the wet.

Effect of High Curing Temperature on Strength of Concrete: (University of California, Berkeley). Tests were made on 2 by 4-in. mortar cylinders of 1:2½ and 1:5 mixes by volume. The curing temperatures were 70, 100, 140 and 180 degrees F.; the time of curing 18, 24, and 36 hours for each temperature, a fog being maintained during this period by means of an atomizer. Some of the specimens were stored dry at 50 per cent relative humidity, and some were stored wet until time of test at various ages. High temperatures increased the early strength of cement mortars, the maximum effect being produced by a temperature of 140 degrees F. maintained for from 24 to 36 hours. In general, at ages of 7 days or more, a curing temperature of 180 degrees F. produced lower strengths than a temperature of 100 degrees F. After the age of 7 days, specimens cured at normal temperatures and stored in water exhibited higher compressive strengths then did similar specimens cured at high temperatures and stored in water.

Further tests are in progress on 3-ft. cubes of concrete insulated on all sides, which indicate to date that mass concrete develops a temperature of about 155 degrees F. within 20 hours when using 9 sacks of cement per cubic yard. The 1-day strength of the mass concrete is much greater than the strength of corresponding 6 by 12-in. cylinders cured at 70 degrees F.

Thermal Expansion of Concrete: (University of California, Berkeley). Tests are in progress to determine variations in coefficient of thermal expansion in which variables are water-cement ratio, richness of mix, type of aggregate, curing condition, and age of specimen. Tests on 3 by 3 by 40-in. bars for temperatures between 15 and 130 degrees F. indicate the following results:

(1) The coefficient of thermal expansion varies with the richness of the mix, the value for a 1:3 mix being about 6 per cent higher than for a 1:6 mix of similar materials.

(2) The coefficient is about 5 per cent higher for water-soaked specimens than for air-dry concrete.

(3) There is no appreciable variation in the coefficient for ordinary ranges of temperature.

(4) The variation in the coefficient for mixes of various consistencies is negligible.

(5) The type of aggregate has a marked influence on the coefficient, the actual values for 1:4½ mixes of aggregates of identical gradations being as follows:

Quartz	0.0000066	per 1 degree F.
Sandstone	0.0000065	" "
Gravel	0.0000060	" "
Granite	0.0000053	" "
Basalt	0.0000048	" "
Limestone	0.0000038	" "

(6) The coefficient of expansion for air-dry concrete cured normally for 2 months and then subjected to 100 alternations of temperature between 15 and 130 degrees F. was decreased about 14 per cent from the value obtained for concrete not subjected to alternations of high and low temperatures, but stored at a constant temperature of 70 degrees F.

Variations in Temperature in Plain Concrete Slabs as Affected by Type of Curing: (Texas A. and M. College, College Station). Investigations have been started to determine variations in temperature within concrete slabs. Tests are to be made in the field, and temperature records taken with resistance wires. Laboratory tests will be made to parallel the field investigation, using the same materials in 6 by 6 by 32-in. beams and 6 by 12-in. cylinders.

TESTS AND TEST METHODS

Effect of Removing the Water that Gathers at the Top of Mortar or Concrete Soon After Placing: (University of Colorado, Boulder). Comparison specimens were made in water-tight molds. As the water gathered it was carefully removed by blotters from some of these. The amount removed was determined by weighing the specimens at intervals. Except in the case of very rich mixtures and a great excess of mixing water, the water that gathered at the tops of the specimens that were not blotted returned within the mass after a few hours. All specimens were stored in water after one or two days. Those which had been blotted absorbed more water during the early storage period but the strengths of the two classes were practically identical. The shrinkages were equal and it is evident that the removal of the water did not add to the amount of settlement (decrease the voids) and that the effective water-cement ratio was not changed from what it was for the specimens from which no water was removed. These results would indicate that it is incorrect to use a corrected water-cement as a measure of leakage effect unless added compaction results from the loss or removal of the water. It has been demonstrated that any method of water removal that does cause the particles to move closer together will lower the water-cement ratio and increase the strength and density.

Variations in Strength of Concrete Cylinders Due to Rodding: (Kentucky State Highway Department and University of Kentucky, Lexington). The purpose of these tests was to determine variations in strength of cylinders not rodded in accordance with standard methods. The only variables permitted in the investigation were the methods of rodding and placing, which consisted of: (a) use of standard rod with 4-in. layers of concrete varying the number of roddings; (b) use of standard rod with 25 roddings varying the thickness of the layers; (c) 4-in. layers rodded 25 times with a variation in size of rod from

¼-in. round to a 2 by 4-in. wooden rod; and (d) placing without rodding or tamping, but pushing down with trowel. It was found that (1) standard rodding produced satisfactory results both as to strength and density, while (2) concrete thoroughly rodded with a ⅝-in. or larger rod, total roddings being as many as 75 in some cases, gave satisfactory strength, but was less dense than concrete rodded in accordance with standard methods. It may be concluded that if reasonable care is used in tamping concrete into the mold and sufficient tamping is done, use of a standard rod is not important.

Relationship between End Condition and Strength of Concrete Cylinders: (Kentucky State Highway Department and University of Kentucky, Lexington). Studies were made to determine the value of true end condition and the most effective way of obtaining it. Tests were made at 28 days on specimens essentially alike except for end conditions, which varied from true to $\frac{3}{16}$-in. concave and $\frac{3}{16}$-in. convex, in addition to one set which had 14 per cent end area chipped off. Cement mortar and sand bearings were used. The following are the outstanding conclusions drawn from the results: (1) the end condition of a cylinder should be carefully noted upon its arrival at the Laboratory, and if convex should be rubbed to a plane surface or capped with a suitable cap. In no case of this type should sand bearing be used; (2) if the end is concave, it can be rectified by rubbing down, or a thin sand bearing of confined sand, not to exceed ⅛ in. can be used; (3) if the specimen is slightly irregular, as in the case of finishing with a trowel, it should preferably be rubbed down to a smooth surface, though a thin sand bearing can be used; (4) reduction of 14 per cent in end area has no effect on strength obtained; (5) if the true value of a concrete is to be obtained from the cylinder it must be made with perfect bearings on each end. Fair to good results may be obtained with the sand bearing or added cap, but neither of these methods are reliable, and it is important that the ends of the cylinders be correctly finished. This laboratory has recently built and installed a cup grinding machine for finishing cylinders, but results from the use of this machine are not yet available.

Effect of Size of Cylinder on Strength of Concrete: (University of California, Berkeley). Cylinders varying from 2 to 12 inches in diameter and two diameters in length were made from an identical mix, using ½-in. maximum size of aggregate. One group of specimens was stored in air, while a second group was coated with Inertol and stored in a continual fog. Variations in moisture content of test specimens of various sizes were the cause of considerable variations in strength. The strength was not materially influenced by size of test cylinder if all other conditions, including the percentage of moisture in the specimen at time of test, were maintained constant, and if the maximum size of aggregate was small with respect to the diameter of the cylinder. With ½-in. aggregate the strength was not influenced, provided the diameter of the cylinder was not less than 3 inches (6 times the maximum size of aggregate).

Effect of Size of Cylinder on Strength of Concrete and Mortars: (University of Colorado, Boulder). Several hundred of comparative tests have been made

on 2 by 4-in., 3 by 6-in., and 6 by 12-in. cylinders of mortars and concretes in which the aggregates did not exceed ¾ in. (¼ diameter of 2-in. cylinder).

For similar curing and test conditions, the variations between specimens of different sizes is no greater than between companion specimens of the same size. Lateral extensometers and longitudinal compressometers and semi-autographic recording devices have been developed and used in the majority of these tests. The stress-strain curves, both lateral and longitudinal, are similar throughout. Recognition of this as fact opens up a fruitful field for research by smaller laboratories in the realm of one and two-man projects in which studies can be made on results from the smaller sizes of specimens. These tests included varied curings and test conditions and for these cases the agreement is much less perfect because of the differences in the rates of drying out, etc. for the different sizes of specimens. A number of tests were made on 2 by 4-in. specimens in which the ¾-in. maximum size of coarse aggregate particle was exceeded, particles up to nearly 2-in. being included in some cases. For such specimens the strengths were much more erratic but the maximum strengths attained were about the same as those from the same concrete in larger specimens. The minimum strengths were much lower, however, depending upon how much such large particles lay in the specimen. The least strengths were obtained when large particles lay on a bias. Failure then occurred by sliding along the inclined bedding plane. Many of these results are in process of preparation for publication.

Study of Methods for Determining the Height of Cores Drilled from Concrete Structures: (U. S. Bureau of Public Roads, Washington). These tests are being made in cooperation with A. S. T. M. Committee C-9 on "Concrete and Concrete Aggregates" to develop a standard way of determining height of cored cylinders.

VOLUME CHANGES

Volume Changes in Concrete Due to Changes in Moisture Conditions: (University of California, Berkeley). This is a continuing investigation, the earlier results of which are presented in the 1928 report. A series of specimens was made using various types of aggregate but for which all other factors such as gradation, cement factor, etc. were constant. Tests show that volume changes are approximately the same for concrete having limestone, quartz and granite as aggregates. Gravel and sandstone concretes exhibit volume changes about twice as great as for the aggregates just mentioned.

A second series of tests of gravel concrete, using 1:4 and 1:6 mixes and water-cement ratios of 0.98, 1.07, and 1.16 for each mix show:

(1) Contraction in air is greater for mixes of high water cement ratio than for those of low water cement ratio, the effect being most pronounced for the rich mixes,

(2) In general the expansion in water is greater for the mixes of low-water cement ratio, but the difference is not marked.

Volume Changes in Concrete Due to Changes in Moisture Conditions: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation

and Arch Dam Committee of Engineering Foundation). These tests consisted in length measurements taken at intervals on 3 by 3 by 40-in. beams (similar to those used by Davis) and were obtained for the mixtures of both the Stevenson Creek and Gibson models for various combinations of air and water curing. In general the results accord with previously reported findings by Davis and others.

Volume Changes in Concrete: (University of Michigan, Ann Arbor). This investigation was started in 1902 to determine the changes of volume occurring in neat cement and rich mortar bars at various ages under different conditions. Results obtained at various periods have been published in current engineering literature.

Volume Changes of Neat Cements: (Lehigh Portland Cement Co., Allentown, Pa.). This is a continuation of a project reported in 1928 in which data are continually being obtained on the behavior of test bars under various conditions of exposure. Numerous interesting relations have been disclosed or indicated between behavior of bars under normal and accelerated tests and the constitution of the cements used. In this connection, it may be mentioned that a small conditioning room maintained constantly at a temperature of 70 degrees F. and 50 per cent relative humidity affords a means of comparing shrinkage of bars under definite storage conditions. The indications thus far are that the majority of "normal" cements are much alike in their shrinkage characteristics, but enough exceptions are found to warrant extensive studies into the causes and possible control.

Volume Change of Concrete: (Kansas State Agricultural College, Manhattan). The purpose of this investigation was to determine the reason for the relatively greater number of shrinkage cracks which appear in sand or sand-gravel concretes as compared with those in concrete in which both fine and coarse aggregates are used. Readings are being taken at definite periods on 151 specimens (4 by 6 by 24-in. beams), 86 of which are over one year old, and only 30 of which are less than 100 days old, to determine the effect of the following factors on volume changes:

(1) Type of coarse aggregate,
(2) Variation of material used as total aggregate,
(3) Type of sand in mortar mix,
(4) Type of sand in concrete mix,
(5) Quantity and type of cement,
(6) Variation in water-cement ratio,
(7) Use of various admixtures.

Data from this investigation were published in a paper by Scholer and Dawley on "Volume Change of Concrete," in the Proceedings of the American Society for Testing Materials for 1930.

Study of Volume Changes: (Research Laboratory, Portland Cement Association, Chicago). Various factors affecting volume change of concrete are being investigated. These include effect of regaging and remixing, size and shape

of specimen, absorptive form, curing, coatings, admixtures, fineness, type and storage of cement, cement content, paste ratio, water-cement ratio, type and kind of aggregates, ratio of fine to coarse aggregate, moisture loss and vibration during mixing. The studies to date have brought out the inter-relation of some of the principal factors involved in these changes and a method has been evolved for estimating the relative shrinkage of various concrete mixtures.

WEAR

Relationship between Cement Factor and Wear and Strength of Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). The purpose of this investigation was to establish an economical cement factor as a basis of control for pavement concrete. A concrete mix was used in which the fine aggregate was 1.2 times the voids in the coarse aggregate, with a cement factor varying from 1.5 to 1.7 bbl. per cu yd. Comparisons were made of different aggregates in beams, cylinders, and 9-in. balls. Below is a summary of the tests made using stone and gravel:

Cement bbl. per cu. yd.	*Coarse Aggregate*	*Water-Cement Ratio*	*Wear Concrete Balls* %	*28-Day Strength*		*Strength Comp. Specimen at 42 Days*	
				Compressive	*Transverse*	*Cylinders*	*Cores*
1.5	Stone	0.74	19.4	3606	860	5061	4888
	Gravel	0.70	19.4	3206	643	3732	3300
1.6	Stone	0.70	17.5	3273	817	4465	4690
	Gravel	0.63	15.9	3899	712	3856	4300
1.7	Stone	0.68	15.0	4361	912	4521	4725
	Gravel	0.60	11.7	3585	692	3990	3990

The above data indicate that 1.5 bbl. of cement per cu. yd. of concrete with 5¾ gal. of water using aggregate that passed a standard specification will give sufficient strength both in tension and compression to insure good concrete pavement.

There seems to be no good reason why concrete made with crushed stone should require from 0.1 to 0.2 bbl. more cement than that made with gravel as is the case with the 1:2:3.5 mix commonly used.

Workability of Concrete: (Ohio State University, Columbus). As a means of standardizing concrete from central mixing plants, tests are being made to develop a satisfactory workability meter, after which a thorough study will be made in an effort to determine why some mixtures of the same proportions are harsh and others are smooth.

Workability of Portland Cement Mortars: (National Bureau of Standards, Washington). This is a continuation of the study described in the paper on "Cement as a Factor in the Workability of Concrete," which appeared in the 1928 Proceedings of the A. C. I. in which the effect of various portland cements on the workability of mortars is described, as well as a discussion of a means of measuring this workability. Further tests have been made for measurement

of workability as described in the foregoing paper, but at present full attention is being given a modified McMichaels viscosimeter, using various mixes with various water content, as well as various gradings of sand.

4—Reinforced Concrete

BOND

Bond Resistance of Reinforced Concrete Beams under Continued Load: (T. & A. M. Dept., University of Illinois, Urbana). A group of 24 beams were loaded continuously over a period of several months by means of jacks and heavy, calibrated spiral springs to determine the effect of continued loading of various intensities on bond-slip relations. Slips and strains were measured at various points in the beams at regular intervals of time. There was a total of 40 beams in all, together with auxiliary compression and pull-out specimens.

BRIDGES

Study of Behavior of Reinforced Concrete Arches During and Immediately Following Construction: (State Highway Board of Georgia, East Point). This investigation was conducted on an arch bridge near Eatonton, Georgia. Measurements were taken to determine the initial behavior of an open-spandrel reinforced concrete arch rib. Readings were first taken during placing of the concrete in the arch ribs and continued throughout the construction of the bridge and for a month thereafter, a total period of about 4 months. The work included measurement of strain in steel and concrete, beginning with zero strain while the concrete was being placed; determination of the compressive strength, modulus of rupture, and modulus of elasticity of the concrete used in the arch rib at various ages; measurement of the vertical movement of the arch rib at the crown, beginning just after the last section of the rib had been completed; measurement of the movement of abutments; measurement of temperatures at various points in the arch rib; and measurements of movements in expansion joints in the decks. Full details of this investigation were reported in the Proceedings of the American Society of Civil Engineers, page 2279, November, 1929.

Tests on Arlington Memorial Bridge: (National Bureau of Standards, Washington). Tests on one of the arches of the Arlington Memorial Bridge are being made by the National Bureau of Standards in conjunction with the Arlington Memorial Bridge Commission to obtain data on the structural behavior of the arch during and after its construction. Arch span No. 7 selected for testing purposes is 177 ft. 4 in. in clear span, and has a rise from springing line to crown intrados of 26 ft. 8 in. Construction of the arch was begun in October, 1928, and it is now completed, with the exception of the roadway and sidewalk paving surfaces.

The temperature of the concrete in the arch barrel has been measured at intervals since the concrete was placed. Other measurements include the deflections and strains of the arch barrel caused by different temperature and load conditions. The concrete temperature has been measured with resistance thermometers and thermocouples, the thermometers being connected to a

potentiometer recorder for sixteen stations. Deflection observations are obtained by means of clinometers which are used on a continuous line of 5-ft. stations extending over the back of the arch and a portion of each supporting pier.

Strain measurements are made at selected points, strain gages being used on the extrados, and electric telemeters buried in the concrete are used at the intrados. The measurements will be continued until data have been obtained on the effects of temperature changes on the deformations of the arch.

Stress Reduction in Concrete Arches by Means of Hydraulic Jacks During Construction for Adjusting the Axis to its Most Favorable Position: (U. S. Bureau of Public Roads, Washington, in cooperation with the Oregon State Highway Commission). Studies are being made in accordance with Freyssinet's method of reducing stress in concrete arches by means of hydraulic jacks.

Investigation of Reinforced Concrete Columns: (American Concrete Institute Committee 105). An extensive investigation on reinforced concrete columns is being carried out at Lehigh University and at the University of Illinois by Committee 105 of the American Concrete Institute. The purpose of this study is to establish the fundamental relationships between load, elastic properties and ultimate strength of reinforced concrete columns as influenced by variations in size, quality of concrete, curing of concrete, amount, arrangement and quality of reinforcement, fireproofing, rate and method of loading and the effect of continuous loading over long periods of time.

The original program includes a total of 531 reinforced specimens and 64 unreinforced specimens. To this number an additional series of 24 specimens has recently been added.

The size of specimens ranges from 8 in. in diameter by 5 ft. long to 32 in. in diameter by 17 ft. 6 in. long. Most of the specimens, however, are of the smaller size. Concrete strengths vary from 2000 to 8000 pounds. per square inch. Sixty are being held under design loadings over a period of one year during which time periodical strain gage measurements are being made to determine the amount and rate of time yield. Measurements are made on companion specimens not under load at corresponding intervals to determine shrinkage effects.

In 4 out of a total of 6 series comprising the original program the work of the two laboratories is intended to be identical. The larger columns are all being tested at University of Illinois where a 3,000,000 pound hydraulic testing machine has recently been installed. Some of the specimens will utilize this machine to practically its full capacity.

Effect of Stressing Steel Encased in Concrete Above Its Yield Point: (University of Colorado, Boulder). Plain and spirally reinforced 6 by 12-in. concrete cylinders were cast each with a single ½-in. square structural steel bar along its axis. The bars had milled ends and projected about ⅛-in. from each end of the cylinders. This required that a bar be stressed to the yield point of the steel before any load bore directly on the concrete. In some specimens the bars were greased and in others not. There were also controls of plain concrete, and spirally reinforced specimens with no bar. Still other

specimens had the bar ground flush and in others the bar did not extend quite to the ends of the specimens. Both lateral and longitudinal strains were measured. While the findings are not all decisive, two facts were demonstrated in so far as the conditions of these tests are concerned.

(1) The plain concrete specimen with bar projecting, but without spiral, split about the time that the yield point of the steel was reached.

(2) The spirally reinforced cylinders with projecting bars did not split. The bar shortened and became flush with the specimen within its yield point range and without visible damage to the concrete. Finally the bars buckled within the specimen after the maximum load was passed and the concrete badly punished.

These tests were run as a reconnaissance of what might be expected in a concrete column if the yield point of the steel is exceeded due to shrinkage, flow, or other cause. The tentative conclusion is that the result should not be serious in a well-spiralled column but it is to be expected that splitting would occur at the yield point, if not earlier in a tied column or a block of plain concrete, in which similar steel was imbedded.

REINFORCING STEEL

Design of New Type of Steel Reinforcing for Large Concrete Conduits: (Iowa State College, Ames). This investigation has been under way for some time and a patent has been secured on a new steel reinforcing. Tests were made at the Indianapolis plant of the Independent Pipe Company on several sections of concrete pipe reinforced with this special steel. The results of the tests will be available soon.

Investigation of Hooks as Anchorage in Reinforced Concrete: (Washington University, St. Louis, Mo.). The purpose of this investigation is (1) to determine the effect of varying strength of concrete, (2) effect of radius of hook bend, (3) effect of kind of bar (plain or deformed) on hook resistance, (4) effect of shape of hook on anchorage resistance, semi-circular hooks compared with hooks of changing radius, and (4) to develop a more efficient hook than the Joint Committee 8d. hook. Sixty anchorage specimens 8 by 8 by 16-in. blocks have been tested to date, including load-slip studies on two grades of concrete for both plain and deformed straight bars, two sizes of semi-circular hooks, and compound-circular hooks. Slip is measured internally, at 2 points on straight rods, and at 2 points of tangency on all hooks, by means of Ames dials attached to indicator rods which are welded to the reinforcing bars. The results obtained thus far indicate that (1) the carrying capacity of hooks tested varies directly with compressive strength of concrete; (2) amount of slip varies inversely with compressive strength of concrete; and (3) compound-circular hooks are more efficient than semi-circular hooks. It is proposed to make an experimental analysis of the load-slip relation, including such factors as stiffness of hooks, sliding friction, and distribution of steel stress. Tests are also planned in which the above forms of anchorage will be used in beams.

5—SUGGESTED RESEARCHES

CEMENT

Development of satisfactory method for making accurate particle size separations of relatively large quantities of cement.

A satisfactory method for determining the actual diameter of finely divided particles.

Determination of particle size and size distribution of finely divided materials including cement. Should include study of air analyzer, Noebel elutriator, microscopic projection, or other methods.

Effect of hot cement on temperature and behavior of fresh cement on properties of hardened concrete.

Use of high strength cement in tests of sand.

Review and comparison of foreign and American methods of testing cement.

AGGREGATES

Development of a reliable test for absorption of fine aggregates.

Effect of organic material in fine aggregates on resistance of mortar and concrete to freezing and thawing.

Effect of surface texture or mineral composition of aggregates on transverse, tensile, and compressive strength of concrete.

Effect of heating aggregates and water on strength of concrete.

Volume change of different types of aggregate when subjected to wide ranges in moisture content and in temperature.

Investigation of properties of cinder concrete; influence of cinder aggregate on corrosion of reinforcing.

Methods of determining durability of fine and coarse aggregates.

Methods for testing the quality of sands for use in concrete (shale content, strength, and resistance to weathering and abrasion, etc.)

Effect of coatings on coarse aggregate on strength of concrete.

Effect of grading of fine aggregate on properties other than strength of mortar and concrete.

Effect of aggregate size upon strength, Young's modulus, Poisson's ratio and other properties, with special reference to the larger gradings such as those which result when cobbles are used in mass concrete.

PLAIN CONCRETE

Effect of vibration during setting on strength of concrete.

Effect of temperature at time of test on strength of concrete.

Effect of calcium chloride and sodium chloride on hardened concrete and methods of protection against their action.

Effect of long-continued mixing on strength of concrete.

Influence of high pressure steam curing on strength, volume change, and resistance of concrete to corrosion in sulfate solutions.

Study of fatigue of concrete under (a) slowly applied loads, (b) impact, (c) high frequency vibration.

Effect of high and low temperatures on strength of concrete at different ages.

Investigation of the uniformity of concrete in deep sections when wet mixtures are used.

Study of the influence of moisture content at time of test on flexural, compressive, and tensile strength of specimens of different size.

Strength relations for different types, shapes, and sizes of mortar and concrete specimens at different ages.

Development of methods of making absorption tests of concrete products.

Study of the durability of finish coat stucco mixtures.

Effect of admixtures such as inorganic silt or clays, diatomaceous earth, etc., on strength, bond, permeability, workability, yield and resistance to weathering of concrete.

Temperature in concrete slabs as influenced by curing methods.

Effect of various factors on volume changes due to (a) temperature, and (b) moisture changes.

Study of deformation or flow of concrete under sustained load.

Tests to determine workability of concrete.

Study of effect of admixtures on workability of concrete.

Simple method of designing concrete mixtures which will recognize that the "character" as well as the grading of the aggregate affect the strength of the concrete.

Laboratory tests for determining the resistance of concrete to weathering.

Effect of percolating water on hydraulic structures.

Study of maximum temperatures attained near centers of concrete masses.

Extent and effect of shrinkage cracks in arch dams.

Air deterioration of concrete in different humidities.

Moisture gradient through masses of concrete.

Magnitude of segregation encountered when concrete is discharged from paving mixer.

Study of internal friction in concrete.

Development of a practical consistency regulator.

Study of causes of scaling of concrete.

Field study of joints in road slabs to determine types least susceptible to spalling due to curing. Particular attention to be given to behavior of dummy joints.

Study of methods of controlling movement of slabs—similar to rail creep.

Determination of that grading of sand and gravel which when used as concrete aggregates and designed by the water-cement ratio method will give the least cost per cu. yd. of concrete.

REINFORCED CONCRETE

Study of the effect of coatings and admixtures on the bond resistance of concrete.

Investigation of the initial stresses developed in steel and concrete due to shrinkage, and their effect on the stresses in flexure.

Study of shearing stresses in webs of shallow beams and methods of web reinforcement.

Effect of mortar coating on reinforcing bars on bond resistance.

Effect of repeated and continued loads at high stresses (1) on bond, and (2) on direct compression.

Bond tests of high early strength concrete.

Initial stress developed in steel and in concrete due to setting.

Effect of curing water on reinforcing steel and depth to which such water permeates reinforced concrete members.

Study of column splices, tied, spiralled, combination, and composite.

Bond—hooks vs. straight bars, radius of hook, etc.

Study of behavior of reinforced concrete arches during and immediately following construction.

Shearing stresses due to concentrated loads on floor slabs.

Effect of concentrated load on a cantilever slab with particular application to most economical design of bridge of width just sufficient to permit passing of two vehicles.

Readers are referred to the JOURNAL *for May, 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931.*

Discussion of Paper by Lars Jorgensen

Memorandum on Arch Dam Developments*

BY FREDRIK VOGT†

The author is to be congratulated for his pioneer work on the development of arch dam construction.

As to the safety of single span arch dams, the records are rather convincing: No one has failed, while a great number of dams of other types have failed for different reasons. It may be objected that the Moyie River and the Lanier Lake arch dams failed,[1] but what the records really tell, is, that the proper arch dams did not fail in spite of the failure of a gravity abutment wing in one case and of a faulty rock abutment in the other case. The lesson is therefore not that the arched structure was weak, but, on the contrary, that in this case it was strong enough to carry the water-load even though the regular support suddenly broke away for half the height at one side.[2] In addition, the failure repeats the old lesson, that a gravity dam on a faulty foundation is a dangerous structure.

Some problems in dam construction are common for all types of dams, which is especially true for the problem of how to make permanent and impermeable concrete. It may in this connection be worth while to refer to the comprehensive investigations carried out by the Norwegian Engineering Society on disintegration of concrete in dams, particularly when exposed to water with small silt content, but high acidity, as very often is found in high mountain and sub-arctic climates.[3] The self-tightening of the concrete referred to by the author, and well known for dams

*A. C. I. *Journal*, Sept., 1930; *Proceedings*, V. 27, p. 1.
†Docent at the Norwegian University of Technology, Nidaros, Norway.
[1]*Eng. News Rec.* 1926, II., p. 616.
[2]*Transact.* A. S. C. E., 1929. p. 1273.
[3]Den Norske Ingeniörforenings Betongkomite *Meddelelse nr. 1*, Oslo, 1930.

in southern climates, can not be counted upon when it is exposed to this type of water. On the contrary, all seepages are then slowly increasing and the cement correspondingly dissolved, if seepages exist.

In addition to this problem, the arch dams have their own problems, namely the analysis of stresses and the design of the dam according to this analysis. Not many special problems in civil engineering have been so much discussed during the later years, as the analysis of stresses in single span arch dams. Most of the investigations, however, have been merely mathematical analyses, and the problem goes deeper than to a clever application of mathematics. A mathematical analysis is indispensable, but it can only give the consequences of certain assumptions, and positive information on what assumptions have to be made is still more indispensable. It is a well known fact that tensile cracks can be observed in structures where the usual theory calls for compressive stresses, and also that some of the earlier mathematical studies on arch dams indicated unavoidable horizontal cracking due to vertical tension at places where no cracks or indication of cracks are observed. If something is wrong, it is hardly the mathematics in itself, but the assumptions on which it is based.

Two main problems, which can not be said to be conclusively solved at present, are: (1) What is the time effect in continuously stressed concrete, and, (2) What are the shrinkage and swelling (volumetric changes) in concrete of dams, including volumetric changes due to chemical changes, temperature changes, and moisture changes, and how are these volumetric changes distributed over the section.

As to the time effect, or flow, much basic information is obtained particularly by Professor R. E. Davis[4], and his results are confirmed by tests at the University of Colorado.[5] The main result seems to be that the flow of concrete under continuous load is about proportional to the stresses, at least up to twice the working stresses. Hence the flow does not change materially the distribution or magnitude of the stresses due to load, but it

[4]"Modulus of Elasticity and Poisson's Ratio for Concrete," *Proc.* Am. Soc. Test. Mat., 1929, Part II.

[5]Gilkey and Vogt, discussion on this paper, loc. cit.

reduces the magnitude of the stresses due to volumetric changes.[6] For wet concrete this reduction seems to be about 40 per cent for slow changes, for dry concrete the reduction is much greater. These results, however, need to be confirmed, and the tests need to be extended to different kinds of concrete and stone under different conditions. Also the effect of variable loads should be investigated, before the problem can be characterized as solved.

Laboratory tests have given information on the volumetric changes which can be expected under certain conditions, but before these results can be used in the analysis of large dams, it is necessary to know the conditions in the interior of the mass. The temperature changes in the interior can be estimated with fair accuracy, but the moisture changes differ considerably from one case to another, hence also the actual magnitude of the shrinkage is variable. In a great number of dams the writer has found variations in shrinkage from 0 to 0.02 per cent according to type of concrete, method in concreting, and climate. Before a stress-analysis can be claimed to be fairly accurate, this matter has to be investigated very carefully. The writer has tried to compile information on this subject,[7] but a cooperative work by all interested parties is needed before the matter can be solved.

The author has introduced grouting of the joints in order to eliminate at least partly the effect of the volumetric changes. If the reservoir is filled before grouting, the dam is exposed to shrinkage stresses, and if the joints are grouted too early, the subsequent shrinkage will produce shrinkage stresses in the final state of the dam. For large dams the radiation of the initial heat demands several years, and if the dam must be put in use before the radiation is completed, the choice of a proper time for grouting is a problem which only can be solved by means of intimate knowledge of the volumetric changes. Furthermore, changes in temperature and moisture content, and in the distribution of these, go on with the times, and cause volumetric changes, which can not be counteracted by grouting. The problem of volumetric changes should therefore not be neglected because of the grouting.

[6]Gilkey and Vogt, discussion "Plastic Flow in Concrete Arches," *Proc.* A. S. C. E., 1930, p. 1440.

[7]F. Vogt "Shrinkage and Cracks in Concrete of Dams" to be printed in English this winter in *Det Kgl. Norske Videnskabers Selskab*, Skrifter, Nidaros, Norway. See also F. Vogt "Fundamental Views in the Analysis of Arch Dams," World Power Conference, Berlin, 1930, paper No. 159.

In the later years the Bureau of Reclamation has provided for grouting of the joints in their arch dams. Also the writer has tried to develop the grouting one step farther by introduction of a "multiple pressure grouting system," by which initial stresses in the structure can be introduced in such a way that they partly counteract the stresses from dead load, water load and shrinkage, thus reducing the final resulting stresses[8].

If the arch elements can carry the total water load without being overstressed, an arch dam must be considered as very safe against failure due to loading. However, in many cases the vertical stresses are higher than the horizontal arch stresses, and vertical tension may develop horizontal cracks, which invite the water to attack the concrete chemically and mechanically. Also the investigation of these vertical stresses is therefore of importance.

For dams in narrow canyons the vertical stresses are usually not the limiting factor. For instance, for a preliminary design for Diablo Canyon dam, the writer in 1927, then employed by the author, computed the vertical stresses at the base of the maximum section as 201 — 135 = 66 lb. per sq. in. upstream, and 247 + 291 = 538 downstream compression due to dead load and water load. The final design will show slightly smaller stresses. Assumed that the joints are grouted at low temperature after the major part of the shrinkage has taken place, the volumetric changes will not aggravate the result enough to make these stresses dangerous.

For dams in wide canyons the ratio of radius to height will be larger, and then a vertical tension at the upstream face is harder to avoid. The Stevenson Creek Test Dam is one example of this. The dam cracked horizontally at the base with water level between half and two thirds of the height. Later on it cracked vertically and horizontally at other places. This test dam has exceptionally daring dimensions in order to obtain large deflections and eventually get a test to failure, and a direct comparison with commercial dams is therefore difficult. Some of the most daring dams may be assumed cracked in a similar manner at the base, but for a number of arch dams, conclusions about

[8]See "Hydraulic Engineering," Los Angeles, Dec., 1928.

such cracks made from mathematical analysis are not checked by any positive indications of cracks in the actual structure. This discrepancy between observation and analysis may be due to wrong assumptions with respect to shrinkage and swelling, or due to incomplete analysis, or both. A number of factors, which formerly were neglected in the analysis, have proved to reduce the vertical tension which otherwise could have been expected, and a complete analysis must take into account not only the deformation of arch and cantilever elements due to thrust and bending, but also shear deformation, elastic yielding of the foundation, lateral deformation, and stiffness against tangential displacements.[9] In some cases perhaps also the stiffness against twisting of the elements may be of importance.

The author has several times also pointed at the release in the vertical bending stresses due to swelling at the upstream wet face and the following warping of the dam. This factor will undoubtedly sometimes be of importance, but as a result of observations on dams, the writer has received the impression that this factor has sometimes been overemphasized, and that no release worth mentioning can be counted upon except for the very skin.

The maximum distance, which can be used between the joints if interjacent cracks should be avoided, must necessarily be dependent on the type of the dam, and the author can hardly expect to find "any spacing which he thinks is the best in general" given by a certain number of feet. If the shrinkage were the same and distributed in the same way for a large dam, and for a model of this in scale $1:n$, the stresses would also be the same in dam and model. As an example, may be mentioned that the shrinkage cracks at both abutment joints in Stevenson Creek dam were truly repeated in a mortar model in scale 1:12. If cracks can be avoided in the dam by a maximum spacing a of the joints, then the spacing in the model should be maximum $a:n$. If dams with similar shaped cross-sections were divided in sections by joints with spacing given as a certain fraction of the height for each block, each block would be a model of all the other blocks, and the same shrinkage in all dams would give the same stresses,

[9] "Fundamental Views, Etc.," loc. cit.

i. e., the proper spacing should be a certain fraction of the height. To some extent this rule may be correct, the writer has, for instance, noticed cracks in all sections midway between joints with spacing 30 to 40 ft. on a small arch dam at Twin Lake, Cal., while large dams usually do not crack if the spacing is 50 ft.

The volumetric changes are, in general, greater for small thin-sectioned structures than for thick-sectioned due to more complete drying, and the spacing of the joints should then possibly be increased for large dams more than in proportion to the height. This, however, is in disagreement with the experience. The reason may be that for large thick dams the volumetric changes in the interior mass is much smaller than for concrete near the face, and that cracks are started at the face due to this difference more than due to the restraining from the foundation. Regardless of the size of the dam, a certain maximum spacing should therefore not be exceeded, but this limit will necessarily be dependent on type of concrete and on climatic conditions at the dam-site, and no simple rule will hold for all locations.

One experience is very frequently repeated, namely that contraction joints are continued down to the very foundation by irregular cracks, if the joints initially stopped at a higher elevation. As a rule all joints should therefore be continued directly down to the very base at once, in order to avoid this cracking.

BY W. NIMMO*

Since there is much truth in the old adage that "an ounce of practice is worth a pound of theory," this paper, in which the author sets forth the results of his long experience in the design and construction of arch dams, is of particular value. Little real progress in the determination of stresses in an arched dam was made until Professor Cain published well known equations which have become the basis for much subsequent work. Numerous attempts have been made to analyze the stresses in an arch dam by equating the deflections of arch slices and cantilevers at one or more points, and, while many elegant mathematical treatments have been evolved, few of them are of real practical use.

The trial load method,[1] developed by practical designers, has

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[1] "Analysis of Arch Dams by the Trial Load Method" by C. H. Howell and A. C. Jaquith, *Trans.* Am. Soc. C. E., V. 93, 1929.

been used to a considerable extent for the design of large dams and the method appears to the writer to be the most suitable of the theoretical treatments, and some arch dams in Europe have been successfully designed by somewhat similar methods. The experimental results obtained from models and from the Stevenson Creek dam show that, at least for the conditions obtained during such experiments, the trial load method yields results which are in close agreement with observation. Most of the published deflection curves for actual dams, including those recently obtained from a number of Swiss dams,[2] exhibit the same general characteristics as those derived from the experimental work.

Notwithstanding these results such an experienced designer as the author, while admitting the existence of cantilever action, considers that its effect is so uncertain that he prefers to neglect it in his analysis of stresses and assumes that all the water load is carried by arch action or in the lower portion of the dam by shear. In the Stevenson Creek and Swiss experiments the variation of water level was so rapid that the effects of water soaking and plastic flow are practically eliminated, but this is not so for a dam in ordinary use. Expansion of the upstream face due to water soaking will appreciably lessen the load supported by the cantilevers, but the effect of plastic flow is less certain. Deformation of the foundation, including movement downstream and tilting at the base and spreading of the abutments at the higher levels and also cracking due to shrinkage was clearly in evidence at Stevenson Creek but their extent in a large dam is still a matter upon which reliable data is lacking. The effect of deformation of the foundation will be to diminish the load carried by the cantilever while shrinkage, if it be excessive, may crack the cantilevers and almost destroy their capacity to carry load. Moreover the dams dealt with by the author have been pressure grouted after shrinkage has taken place and, since the full water load on them usually occurs during the summer, the deflections under load will be less than in the case of those dams which have not been pressure grouted, and this will further decrease the proportion of load that can be carried by the cantilevers.

[2] "Deformationsmessungen an Staumauern nach den Methoden der Geodäsie" by W. Lang *Service Topographique*, Berne, 1929.

The dams that have been designed by the trial load method have not, so far as is known to the writer, been pressure grouted and therefore cantilever action may be expected to play a large part necessitating its inclusion in the analysis. The fact that grouting throws more of the work upon the arch is, in the opinion of the writer, a great advantage. In the southern portion of Australia dams are usually filled during the colder portion of the year so that the maximum downstream deflection due to temperature occurs at the same time as that due to load and under these conditions to minimize cantilever action it would be necessary to employ a sufficient pressure in grouting to force the empty dam somewhat upstream.

It would be of great value if deflection measurements extending over several seasons were available for a large arch dam that has been pressure grouted and one of similar type that has not been so treated. It is remarkable that although computation indicates the existence of tension amounting to 289 lb. per sq. in. in the Lake Spaulding dam, no harmful result has been observed.

The author's satisfactory experience with a mat of rocks as an energy dissipator under high overfalls is of interest, but in this country where masonry dams are usually built in sandstone formations, the method would probably not give sufficient protection against wear.

AUTHOR'S CLOSURE

It has been well stated by both W. Nimmo and Fredrik Vogt, that certain difficulties exist, when it comes to deciding upon proper values to use for modulus of elasticity, shrinkage and water soaking effect when designing arch dams.

More tests have been made to find values for the modulus of elasticity and shrinkage than for the water soaking effect. The main reason may be that the distribution of this effect is much more difficult to find than the other properties of the concrete, but it is at least as important, if one desires to attempt to distribute the waterload between arch and cantilever.

If the design has been based upon a certain calculated distribution of the load between arch and cantilever and this distribution is wrong, "It is hardly the mathematics itself, but the assumptions on which it is based" that is wrong, to quote from Dr. Vogt.

The writer prefers to use higher arch stresses and assume that no load goes on the cantilever, except near the bottom. There have been no tests made on commercial arch dams, as far as the writer knows, where it could be shown, that the cantilever took a certain definite part of the load relieving the arch of just that much. The Stevenson Creek tests may show a certain division of load, but the load in this case came on almost instantaneously and besides the proportions of this dam are not comparable with those of a good commercial arch dam.

Not alone is the sustained modulus of elasticity only about half of the instantaneous (at about 600-lb. compression per sq. inch in air), but the E for sustained load is from 60 per cent to 63 per cent higher for wet than for dry concrete according to tests made by Professor R. E. Davis in connection with the concrete in the Stevenson Creek dam (page 213 *Proc.* Am. Soc. of Civil Engineers, May, 1928, Part 3).

An important item for an arch dam is the grouting of the vertical contraction joints at the right time. The writer prefers to wait until the dam has taken load at least once and has cooled off. Unfortunately the owner will often not wait that long, and on a diversion dam very often grouting can not be put off that long. In such cases grouting will only be say 50 per cent effective and some cantilever action undoubtedly will develop.

It is not intended to grout the Diablo Dam until the Spring of 1932. After that time a power plant will utilize the head behind the dam and the water can not be drawn low enough to relieve the load on the structure sufficient for grouting purposes. By that time the heat should be out of the dam body and the structure should have taken its permanent set. Before grouting has taken place, there will undoubtedly be some cantilever action during the winter with reservoir full, this however, the structure will easily take care of.

The spacing of the contraction joints on the Diablo Dam was approximately 75 ft. This has been shown to be too large, since shrinkage cracks have appeared in the four middle blocks, between the artificial joints, extending from near the bottom to within 80 ft. of the crest. One shrinkage crack goes through the

dam body, the others appear, so far, on the down stream face only. Grouting the artificial joints later is expected to practically close these cracks and the load may do the rest.

For overfall dams on somewhat soft rock as mentioned by Mr. Nimmo it will be the best practice for arch dams to create a lake below the main dam and concentrate the overfall as much as possible into this lake.

In conclusion the writer desires to correct a misprint on page 62 line 17 from below. It should say: T. H. Carver was the principal assistant engineer in charge, etc.

The following is a list of Constant Angle Arch Dams built so far with their principal dimensions given. It can be seen, that there is nothing new and untried about arch dams any more, since some of the highest dams in the world are in this list and have so far given good satisfaction.

CONSTANT ANGLE ARCH DAMS BUILT IN THE PAST

1. Lake Spaulding Dam, South Yuba River, built 1912-13; 295 ft. high above foundation; 275 ft. high above river level; 770 ft. span; 192,000 cu. yd. of concrete; upstream radius at crest 440 ft., at bottom, 250 ft.

2. Salmon Creek Dam, Alaska near Juneau, built 1912-13; 168 ft. above river level; 545 ft. span; 52,000 cu. yd. of concrete; upstream radius at crest 331 ft., at bottom 147.5 ft.; thickness at crest, 6 ft., at bottom 47.5 ft.

3. Kerckhoff Dam, San Joaquin River, California, 1919; 108 ft. high; 450 ft. span; 23,000 cu. yd. of concrete; dam designed for 100,000 sec. ft. free overfall.

4. Carmel River Dam near Monterey, California, 1920; 90 ft. high; 270 ft. span; radius at crest 135 ft., at bottom 80 ft. long; thickness of dam 5 ft. at crest, 16 ft. at bottom; 10,000 sec. ft. free overfall.

5. Bullards Bar Dam, Yuba River, California, 1923; 183 ft. high; 440 ft. span; in connection with power plant. Upstream radius at crest 240 ft. long, at bottom 94 ft.; thickness at crest 6 ft., at bottom 43 ft.; 47,000 cu. yd.; 65,000 sec. ft. free overfall.

6. Lost Creek Dam, Lost Creek, California, 1923; 112 ft. high; 380 ft. span; radius upstream crest 200 ft., at bottom 90 ft. long; thickness at crest 4 ft., at bottom 23.5 ft.; 11,000 cu. yd. Spillway free overfall 15,000 sec. ft.

7. Concow Dam, Concow Creek, Calif., Thermolite Irrigation District, 1924; height 90 ft. above stream bed; span 275 ft.; upstream radius at crest 180.6 ft., at bottom 78 ft.; thickness at top 3.75 ft., at bottom 18.9 ft; 1:3:6 concrete; 63,000 cu. yd. of concrete. Spillway 10,000 sec. ft. free overfall.

8. East Canyon Creek Dam, Utah, 1917; 140 ft. high above river level; 190 ft. high above foundation; about 400 ft. span including tangents; upstream radius at crest 98.8 ft. long, at bottom 70 ft. long; crest thickness 5 ft.; bottom thickness 26.25 ft.; 15,888 cu. yd. of concrete. Total overall cost $8.62 per cu. yd.

9. Sun River Dam, Idaho, 1917; 100 ft. high; 250 ft. span.

10. Clear Creek Dam, Washington, 1914-18; 90 ft. high; 180 ft. span; approx. 5,000 sec. ft. free overfall.

11. Manila Dam, Philippine Islands, 1913; 98.4 ft. high; 100 ft. span.

12. La Jogne Dam, Switzerland, 1920; 186 ft. high; upstream crest radius 126 ft.; 250 ft. span.

13. Crocodile River Dam, South Africa Union Irrigation District, 1922; 198 ft. high; 450 ft. crest length; upstream radius at crest, 225 ft.

14. Amsteg Dam, Switzerland, 1921; 105 ft. high; upstream crest radius 62 ft.; 102 ft. span.

15. Gide River Dam, Sweden, 1919; 85 ft. high; 131 ft. span; upstream radius at crest 98 ft; and 72 ft. at an elevation 43 ft. lower down; spillway free overfall.

16. Butte Creek Dam, Calif., 1915; 45 ft. high; 85 ft. span; spillway free overfall..

17. Mormon Flat Dam Salt River, Arizona, 1923-24; height 229 ft. above bedrock; span 326 plus 90 ft. gravity tangent; upstream radius at crest 193 ft. at river level 100 ft.; thickness 8 ft. at top and varies from 20 to 29 ft. at bottom; 43,000 cu. yd. of concrete; 150,000 sec. ft. spillway on gravity section. Total cost of concrete $473,000; 63,000 acre ft. of storage.

18. Gerber Dam, Klamath Project, Oregon; U. S. Reclamation Service, 1923-24; height 70 ft. above river bed, and 82.5 ft. above lowest foundation; span 375 ft.; length of crest along curve 478 ft.; upstream radius at crest 197 ft.; at bottom 155 ft.; thickness 5 ft. at top, and 15 ft. at streambed; 11,880 cu. yd.; 1:2½:5 concrete; 31½ tons of reinforcement; spillway free overfall.

19. Upper Hubbart Dam, Flathead Project, Montana; U. S. Reclamation Service, 1923; height 92.5 ft., above river bed, 113.5 ft. above foundation; span 400 ft.; length of crest along curve 503 ft.; upstream radius at crest 200 ft., at bottom 194 ft.; thickness 5 ft. at top, 22 ft. at bottom; 1:2½:5 concrete; 17,000 cu. yd. of concrete; spillway free overfall.

20. Montejaque Dam, Caduares River, Spain, 1923-24; height 210 ft. above river level, and 273 ft. above foundations; span 208 ft.; length of center line at crest 256 ft.; upstream radius 123 ft. at crest, 72 ft. at base; thickness at crest 10 ft., at base 55 ft.; 35,000 cu. yd. of concrete.

21. Emigrant Creek Dam, Oregon; Talent Irrigation District, 1924; height 100 ft. above stream level; 115 ft. above foundation; span 360 ft.; upstream

radius at crest 165, at base 132 ft.; thickness 5 ft. at crest, and 19.3 ft. at base; 14,000 cu. yd. concrete; spillway partly free overfall.

22. Balch Diversion Dam, Kings River, California, 1926; 100 ft. high above foundation (initial height), ultimate 150 ft.; 300 ft. span; 9,300 cu. yd. of concrete; upstream radius at crest 184 ft. (initial) at bottom 90 ft.; spillway free overfall 40,000 sec. ft.

23. Glines Canyon Dam, Port Angeles, Washington, 1927; 200 ft. high from bottom of foundation to the crest; radius of upstream face at crest 128 ft., at bottom 62 ft.; dam is 28 ft. thick at the base, and 4 ft. thick at top; arch between two gravity tangents 152 ft. long; 16,630 yards concrete in arch; 20,000 cu. yd. total.

24. Big Santa Anita Dam, near Los Angeles; 225 ft. high; 515.0 ft. span; upstream radius at crest 301.5, and 129 ft. long at bottom; thickness at crest 7 ft., at bottom 48 ft.; 76,000 cu. yd. of concrete. Cost $1,211,000, 1927.

25. Urayama River Dam, location near Tokio, Japan, 1921; height 45 ft.; thickness at crest 4 ft., at bottom 9 ft.; radius at crest 48 ft.

26. Lake Cushman Dam, Washington, City of Tacoma; 280 ft. high; upstream radius at crest 200 ft., and 131 ft.; 150 ft. below crest to bottom; span of arch proper 340 ft.; thickness at crest 8 ft., at bottom 45.5 ft.; 85,000 cu. yd. of concrete; finished 1926.

27. Horse Mesa Dam, Arizona; 305 ft. high from foundation to parapet; top upstream radius at crest 251.4 ft., at bottom 82 ft.; thickness at top 8 ft.; thickness at bottom varies from 43 ft. to 57 ft.; stores 245,000 acre ft.; arch span 450 ft.; total span 784 ft.; contains 150,000 cu. yd., 1927. Cost $1,500,000; spillway capacity 150,000 sec. ft.

28. Bucks Creek Diversion Dam, Plumas County, Calif.; 96 ft. high; 420 ft. span; upstream radius at crest 213 ft., at bottom 156 ft.; thickness of dam at crest 6 ft., at bottom 27 ft.; 19,200 cubic yards of concrete; has two gravity tangents, 1927.

29. Canadea Dam, New York; 130 ft. high above river bottom; one gravity abutment arch span 480 ft.; upstream radius at crest 265 ft.; upstream radius at bottom 200 ft., and thickness at crest 5 ft.; at bottom 44 ft.; 50,000 cu. yd. of concrete, 1928.

30. Santeetlah Dam, Tallasse, North Carolina, Aluminum Co. of America; 202 ft. high above foundation; two overflow gravity tangents; arch span 340 ft.; upstream radius at crest 300 ft., at bottom 207 ft.; thickness 10 ft. at crest, and 48 ft. at bottom; 75,000 cu. yd. of concrete in arch portion; 217,000 yards total; total length of dam 1,120 ft., 1927.

31. Pacoima Dam, for Los Angeles County Flood Control; 365 ft. high above river bed; 375 ft. high above foundation; span 560 ft.; upstream radius at crest 340 ft., at bottom 171.34 ft.; thickness of dam at crest 8 ft., at bottom 96 ft.; 215,000 cu. yd. of concrete.

32. Grizzly Creek Diversion Dam, Plumas County, Calif.; 90 ft. high; 420 ft. span; upstream radius at crest 205 ft., at bottom 140 ft.; thickness of dam at crest 6 ft., at bottom 22 ft.; 18,000 cu. yd. of concrete, has gravity tangent; spillway partly overfall.

33. Von Geisen Diversion Dam on Bear River, California; Nevada Irrigation District; 100 ft. high; 365 ft. arch span between two tangents; upstream radius at crest 205 ft., at bottom 184 ft.; minimum thickness 5 ft.; thickness at bottom 22 ft., in the middle of arch. Contract price $157,000, 1928; spillway free, overfall 10,000 sec. ft.

34. Deer Creek Diversion Dam, Nevada Irrigation District; 97 ft. high; 270 ft. span; upstream radius at crest 136 ft., at bottom 98 ft.; minimum thickness 4 ft.; thickness at bottom 12 ft., in the middle of arch. Contract price $52,000, 1928; spillway free, overfall 7,500 sec. ft.

35. Pigeon River Dam, North Carolina; 190 ft. high, plus 10 ft. overflow; 600 ft. span; upstream radius at crest 320 ft., at bottom 244 ft.; minimum thickness 16 ft., just below overflow crest; thickness at bottom 38.5 ft.; Waterville Hydro-Electric Development of Carolina Power and Light Company; 120,000 cu. yd., 1929.

36. Canyon Diablo Dam on Skagit River, City of Seattle; 372 ft. high; span of arch 540 ft., arch butting against gravity tangents; total span approximately 950 ft.; upstream radius at crest 390 ft., at bottom 163 ft.; thickness at crest 16 ft.; thickness at bottom 98 ft.; 316,000 yards of concrete, 1930; overflow on tangents 100,000 sec. ft.

37. The Reeder Gulch Dam for the City of Ashland; 110 ft. high; overflow 415 ft. span; upstream radius at crest 230 ft.; at bottom 150 ft.; thickness at top 7 ft., at the bottom 19.5 ft.; 15,000 cu. yd. of concrete, 1928.

38. Yellow Creek Dam No. 1, Tuscaloosa, Alabama, 1929; 65 ft. high. This dam has horizontal sections of elliptical shape and is symmetrical about the major axis. Semi-major axis 179.5 ft. at crest; 166.5 ft. at base; semi-minor axis 149.26 ft. at crest; 130.62 ft. at base; thickness at crest 5 ft., at base 20 ft.; approximately 7,000 cu. yd. of concrete.

39. The Calles Dam, Mexico, River Santiago, 1929; height of dam 217 ft.; arch with gravity tangents; length of crest 985 ft.; overhang on downstream side at crown 6.66 ft.; radius variable in the same horizontal plane.

40. The Calderwood Dam, Tennessee, 1930; height of dam 230 ft. above foundation; length of upstream radius at spillway level 337.5 ft., 281 ft. at bottom; designed for 200,000 sec. ft. overflow; thickness at top 25 ft., at bottom 48 ft.

41. The Calaveras Dam near Stockton, Calif., 1930; height of dam 160 ft.; arch with gravity tangents; length of crest 1400 ft.; dam 50 ft. thick at base, and 7 ft. thick at top; upstream radius at crest 365.2 ft., at the bottom 204.8 ft.; lowest bid $838,715; 105,000 cu. yd. of concrete; dam overhangs in the middle on the downstream side and at the abutments on the upstream side.

42. The Juncal Dam, Montecito Water District, Santa Barbara County; height of dam 160 ft.; length of upstream radius at crest 196 ft., at bottom 132 ft.; thickness at the crest 6 ft., at the bottom 64 ft., 1930; span 350 ft.; crest length 440 ft.; 34,400 cu. yd. of concrete.

43. Upper Hansen Dam, Los Angeles Flood Control; height of dam 245 ft.; length of upstream radius at crest 190 ft., at bottom 132 ft.; thickness at crest 8 ft., at bottom 62 ft.; arch with two gravity tangents; right tangent spillway 150 ft. long; span of arch portion 265 ft.; left tangent 170 ft. long; 90,000 cu. yds. of concrete; 90,000 cu. yd. of excavation.

44. Dam near Safford, Arizona; 110 ft. high; 130 ft. span; central angle held at about 130 degrees; upstream radius at top 80 ft., at the bottom 14 ft.; dam 2 ft. thick at the crest, and 3 ft. thick at the base at the crown, thicker at the abutments; spillway capacity 1000 sec. ft.

45. Stewart Mountain Dam, Salt River, Arizona; height of dam 212 ft. above bedrock; 1200 ft. long; 117,000 cu. yd. of concrete; radius at top 273 ft.; at the bottom, face is irregular; thickness at top 8 ft., at bottom 33 ft. minimum.

46. Dam at Sautet on the Drag River near Grenoble, Switzerland; 443 ft. high, with secondary dam 180 ft. high below. Dam is designed for unbalanced pressure of 430 ft. (130 meters) head. Crest length 246 ft. Subtended angle held substantially at 90 degrees. Upstream radius at bottom 76.6 ft.; thickness at the bottom 54 ft. Dam contains 74,500 cu. yd. of concrete. Under construction.

47. Ariel Dam on the Lewis River, Wash. 300 ft. above lowest bedrock. Upstream radius at crest 400 ft.; at bottom 50 ft. Dam 93.5 ft. thick at base and 18 ft. thick at crest. Spillway on gravity abutment; capacity 120,000 second feet of water. Arch part of dam 760 ft. long at crest. Approx. 300,000 cu. yd. of concrete in complete dam. Under construction at present.

Discussion of Report of Committee 506

Proposed "Construction Specifications for Concrete Work on the Small Job"

BY H. N. WALSH*

I SUGGEST the insertion of a paragraph to the following effect immediately after the descriptions of the three classes of concrete on page 81:

Where rapid hardening portland cement is to be used, compression tests shall be made in an approved laboratory, to determine the water-cement ratio that will give concrete of the required strength at the age at which the concrete is to be put into use.

My experience has been that the relation strength to water-cement ratio may vary considerably for rapid hardening portlands, even for different shipments of the same brand. Further, as concrete made with these cements is generally put into service after 7 to 10 days curing, and as the strengths at the early ages are most affected by excess water, there is more necessity for careful determination of the proper water-cement ratio when rapid hardening cements are being used.

In the paragraph on page 81 dealing with water carried by sand, a warning might be added that if fine sand is used (being possibly the only fine aggregate economically available), then the job test for water content should be made obligatory. A fine sand may carry as much a 0.5 gallons of water per cu. ft., when appearing to be only slightly damp.

Many architects and engineers might like more detailed description of the consistency than that given on page 85; they would probably welcome reference to the slump test to indicate the approximate consistencies suitable for different classes of work. I, therefore, suggest consideration of the inclusion of a table on the following lines:

If the slump test is used, the concrete for different classes of work shall have the following slumps:

*Professor of Civil Engineering, University College, Cork, Ireland.

Mass concrete, Decking with little reinforcement, Heavy sections of reinforced concrete	2 in.-4 in.
Thin vertical members	5 in.-7 in.
Thin horizontal members	7 in.-9 in.
Roads	1 in.-3 in.

or as otherwise specified by the Architect or Engineer.

I am aware of the dangers attendant on the inclusion of such a list of slumps. Very probably it has been considered and decided against by the Author. Nevertheless I would like to have the matter discussed.

The amounts of the slumps stated above are merely tentative.

BY A. R. ELLIS*

The author of the specifications, so far as I can see, does not define the "small job." Some small jobs are quite important, and in some of them the quality of the materials involved are consequently of greater importance than the quality of materials on some large jobs.

On Page 71, under Article 4, the author provides that the cement shall be "certified by the dealer to comply with all requirements of the standard specifications, etc." Dealer certification in some instances is apt to be influenced by the desire to make a sale.

Frequently, it is the small job contractor who is imposed upon by dealers with cement that has been in storage sufficiently long to cause its deterioration. Some dealers do not know when deterioration, due to age, begins, and again the desire to make a sale is likely to influence covering up this fact, even if it is known.

Considerable stress is laid on fine and coarse aggregate and even that water from the usual drinking supply shall be used and then the door left pretty wide open when it comes to cement.

I would like to see this reference to dealer's certification of the cement omitted from this specification.

BY J. D. STODDARD†

Article 4 provides that cement shall be "certified by the dealer to comply with all requirements of the Standard Specifications of the A. S. T. M." We consider this to be a dangerous clause and

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†Vice-president and treasurer, The Detroit Testing Laboratory.

one which should not be permitted to appear in any specification or building code. While it is intended to apply to "small" jobs only, the word "small" is a very flexible one and depends largely upon the opinion of the individual or possibly the viewpoint of the individual as to whether it should be considered small or large. Our experience with "dealers" has been such that we do not believe any great amount of value should be attached to any certification of the quality of portland cement purchased from them. Very few dealers have more than a smattering of knowledge regarding the nature of portland cement or the conditions under which its integrity is preserved or under which it deteriorates, and certainly the certification of quality should not be placed in the hands of such individuals.

Outside of the commercial phases of our activities which we are of necessity compelled constantly to recognize we are interested in the development of the construction industry along wise and safe lines. We trust you will appreciate our interest in the clause referred to, and hope the language will be changed and no such specification be adopted or recommended for use.

Readers are referred to the JOURNAL *for May, 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931. The proposed specification will be open for discussion at the 27th Annual Convention when the committee will move its tentative adoption.*

Design and Control of Concrete for Diablo Dam*

BY H. F. FAULKNER† AND R. R. HUBBARD‡

DESIGN

The Diablo dam is in a very rugged section of the Cascade Mountains on the Skagit River about 100 miles northeast of Seattle, Wash. It is the second major unit to be completed in the comprehensive development of that river to furnish hydro-electric power for the City of Seattle.

The first unit completed was the Gorge Creek tunnel and power plant, several miles down stream from Diablo. This consists of a 21-ft. diameter concrete lined tunnel driven 11,000 ft. through solid rock, terminating in three penstocks which lead to the three generators in the power house. A temporary crib dam and intake at the upper portal supplies water to this tunnel, which has a fall of 265 ft. in its 11,000 ft.

Diablo Dam is the first permanent dam to be constructed, and is the middle dam of three to be located on 14 miles of the river. Ruby Dam, much larger and higher than Diablo, will be next to be built, 7 miles up stream from Diablo; and later a permanent dam at Gorge Creek, 6 miles down stream, will complete the 1,000,000 H. P. development.

The base of Diablo Dam is in a narrow gorge which the river has cut through a dyke of hard granitic rock lying across the main valley floor. At the dam site the walls of this gorge rise almost perpendicularly to a height of 200 ft., above which elevation the slopes flatten. The average width of the gorge is 150 ft. at the dam site.

The central portion of the dam is of the constant angle arch type, terminating at each end in gravity abutment sections in

*For presentation and discussion 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.
†Assistant Engineer, City of Seattle.
‡Superintendent of Construction, City of Seattle.

which are the spillways. The dam is 390 ft. high, with a crest length of 1170 ft.; has an extreme thickness of 140 ft. at the base and 16 ft. at the crest; and contains 320,000 cu. yds. of concrete. The arch portion of the dam contains 8 sections, each 73 ft. between contraction joints, having keys 6 ft. by 12 in. Three beaded copper water stops are placed vertically in each joint, connecting the adjacent sections. Two of these copper water stops are near the upstream face of the dam, on either side of a 6-in. block or prism of asphalt which was built vertically into the joint during construction and which was melted by electricity when the dam was completed, to effect a final sealing. The third copper water stop was placed near the down stream face of the dam. Perfection grout pipes were provided vertically in each key-way. These will be grouted after the dam has taken its final settlement under load.

At a large bend in the river about one-half mile below the dam site was an extensive sand and gravel deposit known as Reflector bar. Investigations disclosed that this bar would furnish an ample supply of aggregates for the dam and an excellent level site for the large construction camp of the contractors and engineers.

The concrete was designed and specified with particular recognition of the facts that the hydrostatic pressures would be very great, that permeability is in direct ratio to pressure, that any dissolving of the cement by percolating water is proportional to the quantity of percolating water, that it was necessary to obtain concrete of low permeability and that the arch type of dam, of relatively thin section, logically requires a stronger, more nearly impermeable concrete than does one of gravity section.

Samples of pit run material from various parts of Reflector bar were sent to the city laboratory at Seattle for preliminary studies. Tests indicated that there might be insufficient sand and a deficiency of finer particles in the sand available; that there would be plenty of intermediate gravel from ¼ in. to 3 in., with an excess of pea gravel, ¼ in. to ¾ in.; that there was plenty of large gravel and oversized boulders to supply any deficiencies in the aggregate if this larger material were crushed.

It was decided early not to use plums or very large aggregate, since we believe that such aggregate detracts from the work-



FIG. 1—GORGE OF SKAGIT AT DAM SITE



FIG. 2—REFLECTOR BAR—SOURCE OF AGGREGATE

ability or mobility of the fresh concrete mass and does not readily permit uniform placement and the impermeable structure necessary to secure watertight concrete. In the depositing and subsequent plastic settlement of concrete, water, the lightest ingredient, tends to rise. Coming in contact with the undersurface of large aggregate or plums, it can rise no farther but collects there as a film and through it the cement particles that may have been in contact with the aggregate surface, settle or disperse. Such water films create corresponding voids in the hardened concrete and afford ready passage to percolating water. In concrete of fairly stiff consistency such voids may be so thin as merely to cause a weakening of bond, but in wet concrete they may be so large as to be readily visible. It appears to be desirable to eliminate them.

For uniformity of mix it was decided to separate the coarse aggregate into two sizes, and to do all proportioning by weight. It was also decided to fix the maximum diameter of aggregate at 4 in., although permitting the use of elongated particles whose third dimension did in some cases run as high as 8 inches. After screening the samples and giving due consideration to the economic production of the bar, the following grading was specified:

Fine Aggregate: 0 to No. 4 with from 20 to 40 per cent passing a No. 28 sieve		
Medium Aggregate:	Per cent passing 2″	95 to 100
	1½″	55 to 80
	1″	30 to 60
	½″	10 to 25
	¼″	0 to 5
Coarse Aggregate:	Per cent passing 4″	87 to 100
	3″	30 to 70
	2″	0 to 5

With the above grading of aggregates, the concrete mixture finally specified and used in the construction of the dam was determined by trial: various mixes containing various quantities of the different classes of aggregate were made and were compared as to their yield, strength, permeability, and (insofar as the elusive quality can be determined by both slump and by effort required to place properly in a test form) as to workability. Batches containing 20 lbs. of cement furnished sufficient con-

crete for the other tests and finally to make strength cylinders and permeability discs, which latter were 12 in. in diameter and 6 in. thick. The various aggregate proportions were tabulated in terms of their screen sizes and fineness moduli, but it was on the basis of the results obtained with the *trial mixes* that the final decision was made.

After making and testing specimens from the various trial mixtures, one using 20 lbs. cement to 160 lbs. of combined aggregate was decided upon as being most suitable. Making certain allowances for the rather coarse sand (F. M. 3.46) and the elongated shape of the coarse aggregate, and on the basis of the fineness modulus table, the selected mix had the following proportions, by weight:

1.0 parts cement
2.3 parts sand
3.5 parts medium aggregate
2.2 parts coarse aggregate

This mix produced a concrete which with a w/c of 0.75, 87.7 per cent solids and cement content of 1.19 barrels per cubic yard, could be readily placed, had a slump of 3½ in. when the aggregate over 2 in. in diameter was removed, and developed a compressive strength of 3,200 lbs. per sq. in. at 28 days. Best of all, the permeability discs, 6 in. thick, subjected to a pressure of 100 lbs. per sq. in. for 72 hours, remained dry on their unexposed faces for the entire period, such moisture as did penetrate through them being immediately removed by evaporation.

It was thought possible that the contractor might decide to crush the rock excavated from the dam foundations for use as aggregate. Since crushed aggregates do not produce as workable concrete for equal w/c, their use in the amount specified for gravel aggregate, i. e., 1.0 parts cement to 8.0 parts of graded aggregates, by weight, would require the use of more mixing water for equal plasticities, with a resulting loss of strength and increase of permeability. Therefore, it was specified that if crushed coarse aggregate were used, the mix should be 1.0 parts cement to 7.5 parts of combined aggregates, while if both fine and coarse

aggregates were crushed the mix should be 1.0 parts cement to 7.25 parts of combined aggregate.

The above is the "Class A" concrete which was specified for the main body of the dam.

In the upstream face of the dam to a depth of 3 ft. "Class B" concrete was placed integral with the "Class A" concrete. In this, the same gradation and relative proportion of aggregates was used, but the ratio of cement to aggregate was specified as one part cement to 5½ parts aggregate, by weight. This concrete required 1.60 barrels cement per cu. yd., and a w/c of .65 to produce the proper consistency. This richer mixture in the upstream face of the dam is an additional safeguard to a structure which is expected to serve for many centuries. The possibility of differential stresses between the two classes of concrete was considered, but it is believed that any such stresses will be minimized in that the gradation of the aggregate was the same in both classes of concrete.

Some of the principal requirements of the specifications were:

1. A mix of uniform fineness modulus, to be obtained by varying the quantities of the different aggregates.
2. Mixing time of 1½ minutes.
3. Concrete to be as dry as could be properly handled, but not to have more than 3-in. slump.
4. No chuting of concrete to be allowed.
5. No concrete to be dropped more than 3 ft.
6. Concrete to be spread over sections in layers not to exceed 6 in. thick.
7. Concrete not to be deposited in any section to a depth of more than 5 ft. at any one pour.

Some of these requirements were modified as the work progressed and as conditions appeared to warrant.

FIELD OPERATION AND CONTROL

A Telsmith screening and washing plant was installed by the contractor near the lower end of Reflector bar and river aggregate was used for the entire work. Sand and gravel were excavated by a Monoghan dredge and conveyed by a belt to the screening plant. This dredge could not furnish sufficient material at all times and a Sauerman drag line was installed to supplement the supply. The revolving screens at the plant had openings as follows for the production of the required sizes of aggregate:

For sand	⅛ in. by ¾ in.
For medium aggregate	2¼ in. square opening screens
For coarse aggregate	4½ in. square opening screens

Considerable difficulty was experienced at times in obtaining sand which contained the necessary amount of finer particles. The sand grading in the bar varied greatly in different locations. Usually if the dredge were working in coarse sand the Sauerman was placed where finer sand was known to be and vice versa.

Controlling the sand grading was something of a problem. Eight samples were taken at the screening plant from the stream of sand while it was filling a 38 yd. car in which the sand was conveyed from the bar to the storage bins at the damsite. These samples were mixed to form a composite sample, which was dried and screened. Certain tables and rules were worked out governing the acceptance of each car as follows:

If the percentage of sand retained on a No. 28 sieve was under 75, the fineness modulus could go as high as 3.60.

If the percentage retained was 76 to 80, the finess modulus must be 3.40 or under.

If the percentage was above 80, the car was rejected, or, as was the usual case, was held until the cars of finer sand were produced, which, on going into the storage bins at the same time, would bring the average grading within the specified limits.

Medium aggregate was comparatively easy to control. Pea gravel under ½ in. was almost entirely removed into a separate compartment at the plant. A sample from each car was screened and if the fineness modulus was found to be 7.90 to 8.00, 7 cu. yds. of pea gravel were added to the 38 yd. car:

If from 7.80 to 7.90	5.5 yards were added
If from 7.70 to 7.80	4.0 yards were added
If from 7.60 to 7.70	3.0 yards were added
If under 7.60 no pea gravel was added.	

There was no adjustment of the coarse aggregate except by the screens. All material brought to the plant between the 4½ in. and 8 in. sizes was put through a gyratory crusher, which was supposed to produce the 4 in. size, but for two reasons produced more medium than coarse material. These reasons were: first, the opening in the crusher was too small; second, the nature of the rock which, although having a compressive strength of 25,000 lbs. per sq. in. when tested in 3-in. cubes, was nevertheless

brittle, causing it to break into small pieces. The sizes produced with this crushed material, coupled with the plentiful supply of medium aggregate as it occurred in the natural deposit, developed an excess of the medium sized aggregate. Some of this excess was wasted, but much of it was stored and was either used for the reinforced sections of the dam or was sold to another contractor who used it for lining the tunnel between the dam and power house, for setting the penstock, and for building the surge chamber.

An average gradation of material produced is as follows:

	Fine Per Cent	Medium Per Cent	Coarse Per Cent
Ret. on 4″			12.8
3″			61.8
2″		4.1	91.1
1½″		20.0	95.2
1″		36.6	100
¾″		53.0	
⅜″		84.5	
No. 3	0.0	94	
4	2.5	100	
8	12.5		
14	41.5		
28	72.5		
48	93.5		
100	98.5		

The aggregates were loaded into 38 cu. yd. standard gauge cars and were taken to storage bins near the dam. These bins were cribs built on a steep hillside and held about 4,000 yards each of the three grades of aggregate, making the total storage 12,000 cu. yd. The material was drawn off through gates at the bottom of these cribs onto belts which conveyed it to small bins over the weighing hoppers at the mixing plant. Some difficulty was experienced with segregation of the coarse aggregate in the side hill bins, as, on drawing off, the coarser sizes would roll to the bottom, leaving the finer sizes on the slopes. This difficulty was overcome by putting back haul scrapers into the bins, which pulled the aggregate to the bottom with very little segregation.

Bulk cement was delivered to the job in 250 bbl. cars. This was scraped from the cars onto belts which conveyed it to silos

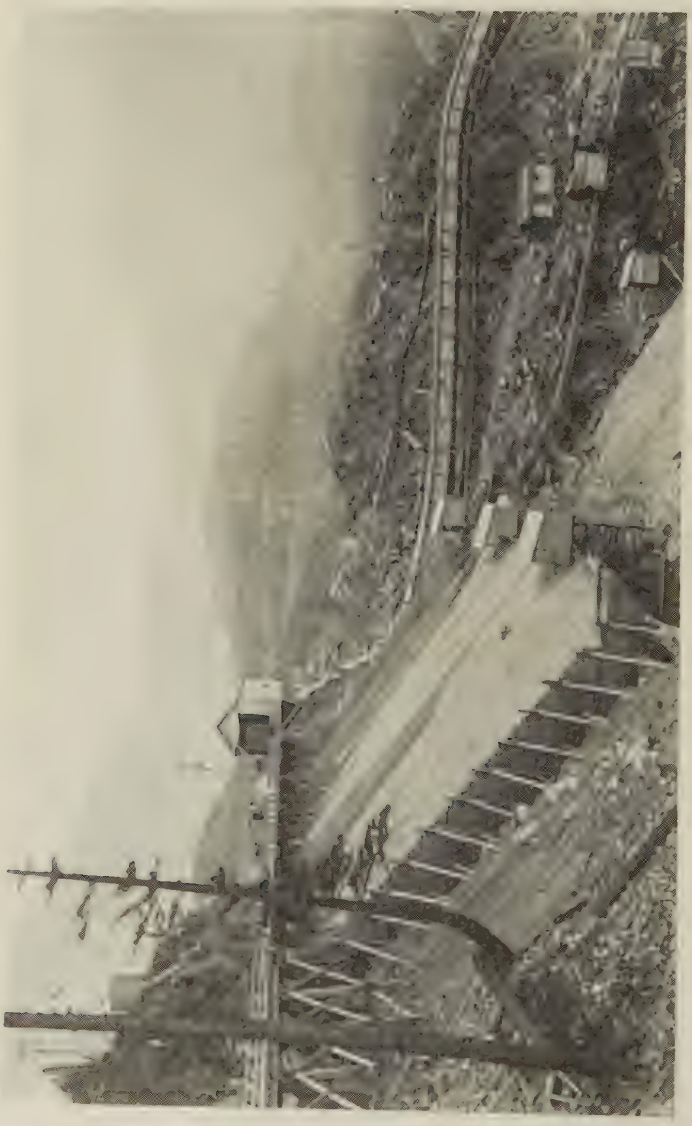


FIG. 3—AGGREGATE STORAGE AT DAM SITE AND BULK CEMENT CONVEYOR

at the mixing plant, from which a screw conveyor delivered it to the cement weighing hopper. Concrete was mixed by two 2-yd. Ransome mixers. Fine, medium and coarse aggregates were weighed in one hopper, three settings of the scale being necessary for each batch.

A typical mix for "Class A" concrete where none of the aggregate was crushed was: 940 lbs. cement, 2260 lbs. fine aggregate, 3142 medium aggregate, and 2090 coarse aggregate. Where a large percentage of both medium and coarse aggregate was crushed, as was frequently the case, a typical mix was 1003 lbs. cement, 2186 lbs. sand less 120 lbs. water, 3200 lbs. medium aggregate, and 2257 lbs. coarse aggregate. Moisture determinations of the sand were made and the entrained water was deducted from the quantity necessary for the proper consistency of the batch. A w/c of from .70-.75 was all that was required to produce a slump of 2½ to 3½ in. The water additional to that in the sand was measured very accurately by two 2-in. Neptune meters. These were accurate throughout the job to the nearest ¼ gal.

For the "Class B" or face concrete the aggregate mix remained the same, the only difference being that the cement content was increaesd to secure the ratio of 1 part cement to 5½ parts aggregate. From the mixing plant on the north rim of the gorge and about 100 feet downstream from the dam, the concrete was discharged into a hopper and thence was chuted down to the base of the dam, where two tall hoisting towers elevated it in skips to other hoppers on the towers located at the zone of construction operations. At the commencement of the concreting operations the concrete was discharged from the skips into 2-yd. bottom dump buckets which were handled by derricks. With the long cables necessary, this process was very slow and unsatisfactory, as it was difficult to position the heavy swinging buckets of concrete properly or to empty them without having the concrete pyramid and pile up in a manner requiring a great deal of shoveling and spreading to secure proper placement.

Later the contractors installed a belt conveyor system on the towers, which worked most efficiently. This installation con-

sisted of two 24-in. belts, the inner one being slung directly from the tower, while the outer one was suspended from a trolley on a steel boom. These belts were adjustable vertically on both tower and discharge ends, as the elevation of placement required. Each belt was driven independently by a small motor. Concrete discharged from the tower hopper onto the inner belt was conveyed outward and was discharged onto the outer belt through a short vertical pipe section which formed the hinge or elbow connecting the two belts. From this elbow the outer belt carried the concrete to its discharge into the "elephant trunk," suspended at the end of the outer belt. These belts were supported in a manner which allowed them to be jack-knifed through an arc of almost 360 deg. By proper jack-knifing of these belts a layer of the required thickness was spread rapidly and uniformly over the entire area of the section being placed, without segregation and requiring very little handling for placement. This conveyor handled as much as 1500 yds, in a 24-hour day. Our specifications required that concrete should be dropped not more than 3 ft., but it was found that better results could be obtained by dropping from 10 to 20 ft. with this "elephant trunk" method of depositing, and it was consequently permitted.

The concrete was of such consistency that ordinarily a man would sink about ankle deep in the fresh mass. During hot weather it was necessary to run the concrete somewhat wetter than a 3-in. slump, and during heavy rains which prevail at the damsite during the winter the consistency was reduced below a 3-in. slump, as water gains would occur as the pour progressed if not guarded against.

After a 5-ft. lift had been made, no more concrete was placed on that section until the concrete had hardened sufficiently to withstand the applied load without injury. Horizontal surfaces were prepared for succeeding pours by vigorous brushing with a stiff brush as soon as the concrete had stiffened sufficiently so that the sweepings were crumbly. The brushing removed the glaze and also any small amount of laitance which might have formed in low spots. Before fresh concrete was deposited the surface was picked where necessary, was hosed off under a high pressure water jet, and was then air blown. Upon this surface a



FIG. 4—UPSTREAM VIEW OF DAM DURING CONSTRUCTION



FIG. 5—DOWNSTREAM VIEW OF DAM UNDER CONSTRUCTION

layer of 1:1½ grout was thoroughly brushed in immediately preceding the concrete deposit.

On each 5-ft. lift a raised key 5 ft. wide and 15 in. high was placed 5 ft. back from the upstream face of the dam. About one foot back from the upstream face a strip of 26 gauge copper 8 in. wide was imbedded 4 in. in the surface of each 5-ft. lift, leaving 4 in. projecting to be imbedded in the next lift. This formed a water stop in the horizontal construction joints.

Concrete was cured by covering and sprinkling horizontal surfaces for 10 days, or until a fresh deposit was placed. The up and down stream faces of the dam were sprayed through perforated pipe hung along the top of the concrete. The ends of sections at the radial contraction joints were painted with asphalt as soon as the forms were removed.

No attempt was made to place concrete when the temperature was below 25 deg. F., as these occasions were rare and the plant was not provided with sufficient steam for protection in cold weather.

A few cracks developed in the dam during construction. They are entirely confined to the lower portion, where the sections are thick, and they occur on radial planes almost midway between contraction joints. None of these cracks has been traced through the dam, and they cease entirely at an elevation where the dam becomes about 40 ft. thick. Since the dam has been completed and filled no evidence has been observed of any leakage through the concrete itself. A few slight leaks have been found which come through the horizontal construction joints. Most of these are in proximity to the abutments and apparently come through cracks in the rock of the gorge wall, the water getting into the construction joints back of the keys and water stops. If keys and stops had been placed along the ends near the abutment walls, this condition would probably not have developed.

About 600 test cylinders of the various classes of concrete were made on this job. These were made by many different inspectors and were cured at the job laboratory in wet sand which was kept at a temperature of from 60 to 70 deg. F. until two days before test date, when they were packed in wet sawdust and shipped to the laboratory in Seattle. From the 6 by 12-in.



FIG. 6—CONCRETE BEING DEPOSITED THROUGH "ELEPHANT TRUNK"

cylinders aggregate coarser than 2 in. was removed; aggregate as large as 3 in. to 3½ in. was allowed in 8 by 16-in. cylinders. Of 350 "Class A" cylinders tested at 28 days:

2	failed under	2500 lbs. per sq. in.
11	failed between	2500 and 2800 lbs. per sq. in.
20	failed between	2800 and 3000 lbs. per sq. in.
53	failed between	3000 and 3500 lbs. per sq. in.
191	failed between	3500 and 5000 lbs. per sq. in.
52	failed between	5000 and 5500 lbs. per sq. in.
20	failed between	5500 and 6000 lbs. per sq. in.
1	failed over	6000

Average	7 day	"Class A" 6 x 12 in.	2662 lbs. per sq. in.
Average	28 day	"Class A" 6 x 12 in.	4297 lbs. per sq. in.
Average	6 mo.	"Class A" 6 x 12 in.	5709 lbs. per sq. in.
Average	1 yr.	"Class A" 6 x 12 in.	5907 lbs. per sq. in.
Average	28 day	"Class A" 8 x 16 in.	3817 lbs. per sq. in.
Average	7 day	"Class B" 6 x 12 in.	3377 lbs. per sq. in.
Average	28 day	"Class B" 6 x 12 in.	5100 lbs. per sq. in.
Average	6 mo.	"Class B" 6 x 12 in.	6350 lbs. per sq. in.
Average	1 yr.	"Class B" 6 x 12 in.	7250 lbs. per sq. in.

Permeability cylinders 8 by 9 in. were made from concrete as used at the dam and were taken to Prof. I. L. Collier at the University of Washington. These were subjected to 170 lbs. per sq. in. pressure and were found to be water tight.

The design and construction of this dam was under the direction of the City Engineer; T. H. Carver, Assistant City Engineer having direct charge. The Constant Angle Arch Dam Co., of San Francisco were consultants; E. D. Alexander was Resident Engineer; and Winston Bros., Minneapolis, Minn., Contractors.

Readers are referred to the JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

Some Long Time Tests of Concrete*

by M. O. Withey†

INTRODUCTION

Two programs of experiments to ascertain the age-strength relationship for concrete cured under common conditions are in progress at The University of Wisconsin. The older test program, herein called Series A,[1] was begun in 1910 and includes crushing tests on 450 6 x 18-in. concrete cylinders of 1:2:4 and 1:3:6 proportions together with auxiliary tests on neat cement and standard sand mortars. Only one set of materials was used in these tests. The three curing conditions in Series A are: 1, storage in a water tank; 2, storage in a cage out of doors; and 3, storage in a cellar. Originally the program was planned to extend over a 10-year period but changes have been made in the test ages to extend the breaking periods to 50 years. Results of tests after 20 years storage are recorded in this report.

In the summer of 1923 a more comprehensive test program, Series B, was initiated. In this, four brands of cement, three coarse aggregates, three proportions, two consistencies, and two curing conditions were used. The total program included 1600 6 x 12-in. concrete cylinders for compression tests, 120 for absorption tests, and 30 for alternate freezing and thawing tests; 96 4 x 6 x 24-in. concrete slabs for expansion tests; 960 2 x 4-in. mortar cylinders, and 1080 briquettes. The curing conditions for this series are: 1. Out of doors as in Series A; 2. In a room in the laboratory.

It is planned to have these specimens tested at 10 intervals over a century. Results of tests up to 5 years are now reported.

*For presentation and discussion at 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.

†Professor of Mechanics, College of Mechanics and Engineering, University of Wisconsin, Madison.

[1]For previous reports on this series of tests see *The Wisconsin Engineer*, Feb. 1915, Feb. 1918, and Nov. 1920.

Acknowledgments

The courtesy of the Wisconsin Granite Company, of Redgranite, in providing the crushed granite used in the tests of Series B is gratefully acknowledged.

Assistance in making the specimens of Series A was rendered by E. B. Nelson, '13; E. J. Paulus, '11; A. Ludberg, '11; and C. M. Scudder, '11. Messrs. S. D. Wonders, '13; C. M. Osterheld, '14; F. D. Bickel, '15; O. A. Bailey, '15; M. B. Hunder, '30; and L. M. Basford have assisted in testing and in preparation of data for reports.

The tests of Series B were made by C. A. Wiepking, with the assistance of R. S. Phillips, both at the time instructors in Mechanics, and W. Laflash, laboratory attendant. Mr. Wiepking also prepared many of the data for this report, A. H. Armstrong and M. B. Hunder assisted.

TESTS OF SERIES A

Materials

Cement—Atlas portland cement from Hannibal, Mo., was used in these tests. A sufficient supply for the entire program was mixed and stored in tightly covered milk cans. One 10-gallon can of this cement was set aside and retested from time to time when tests were made on the concrete specimens. Fig. 1 shows the effects of long storage in the tightly covered can on the tensile strength properties of this cement.

Aggregates—Janesville sand containing about 60 per cent quartz and 30 per cent dolomite was used in making the concrete specimens of the earlier tests. It weighed, when measured dry and loose, 109 lbs. per cu. ft. and contained 34.3 per cent voids. Approximately 97 per cent passed a ¼-in. sieve, 62 per cent passed a No. 30, and 3 per cent passed a No. 100 sieve. The specific gravity of the sand was 2.66.

A rather soft magnesian limestone crushed to pass a 1¼-in. sieve with 64 per cent passing a ¾-in., and 5 per cent passing a ¼-in. mesh was obtained from a local quarry and used in these tests. It weighed 87½ lb. per cu. ft. and contained 48 per cent voids. The specific gravity was 2.74.

Making and Testing Specimens

Making Specimens—The volumes of cement and air dry aggregate for each batch of 15 6 x 18-in. concrete cylinders were weighed; water was measured by a calibrated tank. Materials were mixed in a No. 0 Smith mixer ¾ min. dry and 2¼ min. after water was added. In emptying the mixer and in filling the molds care was taken to insure uniform disposition of the mixed concrete. Metal molds were assembled on leveled steel plates and every mold was filled about ⅓ full of concrete from each of the three wheelbarrows used in emptying the mixer. The concrete was puddled with an iron rod to expel air bubbles. After the concrete had partially hardened, tops of cylinders were capped with a thin layer of 1:1½ mortar.

Neat cement and mortar test pieces were made in accordance with the standard methods used in cement testing in 1910.

Curing—Concrete specimens were kept in molds three days, then marked, measured, weighed, and placed in a moist chamber where they were sprinkled twice a day for eleven days. Five specimens from each batch were subjected to each of the following curing conditions:

1. Cylinders were submerged in a 4½ x 4 x 2¾-ft. water tank at a temperature usually between 60 deg. and 70 deg. F. but occasionally dropping to 35 deg. F. for short periods in winter. During the first two months of storage the water in the tank was changed frequently, and for the first four years thereafter it was changed once a month. Subsequently it was changed at irregular intervals but usually three or four times a year.

2. Cylinders were placed with ends on the ground in an uncovered cage having a northeast exposure. The temperature range for the 20-year period was 25 deg. to 101 deg. F., the average was 46 deg. F. The mean humidity for that period was approximately 75 per cent with occasional departures of ± 10 per cent.

The average annual precipitation at Madison is 32 in. (includes snow). The average snowfal is 38 in. It is estimated that the specimens were subjected to 300 to 350 cycles of freezing and thawing during the twenty-year test period.

3. Cylinders were placed with ends in contact with the ground

in a cellar. The estimated range in temperature approximated 35 deg. to 70 deg. F. and the humidity range 50 to 75 per cent.

The neat cement and mortar specimens were stored one day in the moist closet and 13 days in a water bath at room temperature prior to being subjected to the previously mentioned storage conditions. In the cage and in the cellar these specimens rested on shelves.

One week before testing, concrete specimens were brought into the laboratory and were again weighed on the day broken. Neat cement and mortar specimens were tested immediately after removal from the given storage condition. In the early tests during the winter months frozen specimens were thawed before testing.

The schedule was so arranged that only one concrete specimen from a batch and curing condition was tested at a given time. For each age, mix, and curing condition five cylinders and three or four mortar specimens were broken.

Testing—Rough spots on the ends of cylinders and cubes were removed. In the tests made up to the ten year period two thicknesses of blotting paper were used on the top and bottom surfaces of compression specimens. In subsequent tests they were bedded with plaster of Paris. Speed of testing in the first seven years was somewhat less than 0.1 in. per min. Thereafter, the speeds were 0.06 in. per min.

Briquettes were tested in a Riehlé automatic shot briquette tester, but the roller bearings used in the early tests were inferior to those used in 1930.

Results of Tests

Tests on Stored Cement—The data in Fig. 1 and similar data on the compressive strengths of mortars show that portland cement may be stored for a very long time in a tightly closed vessel without suffering any material change in its strength properties. The changes represented by the data probably reflect discrepancies in testing procedure exercised by the different operators more than variability in properties due to age.

Effect of Curing Conditions on Strength of Mortars—The results of the tensile tests of neat cement and 1:3 standard sand mortar

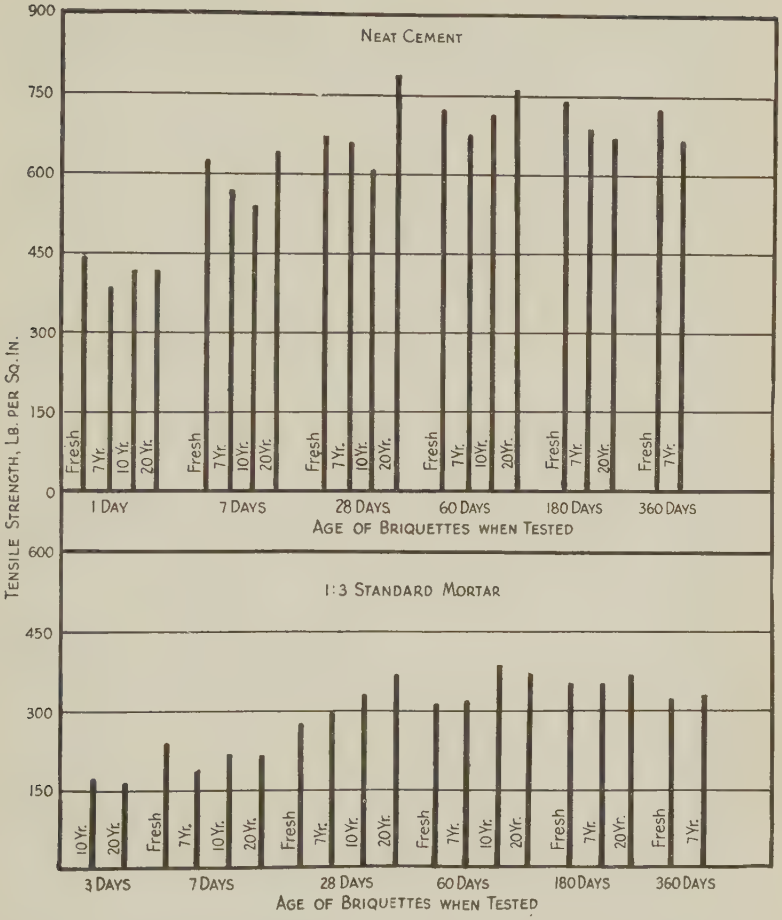


FIG. 1—THE EFFECT OF STORAGE IN A CLOSED MILK CAN ON THE TENSILE STRENGTH OF ATLAS PORTLAND CEMENT

briquets subjected to the different curing conditions are shown in Fig. 2. The neat cement specimens show wide variations in tensile strength under all three curing conditions. Much of this variability was undoubtedly due to surface cracks which were more or less pronounced in many of the neat briquets. The tensile strength of the 1:3 mortar briquets cured in the cage or

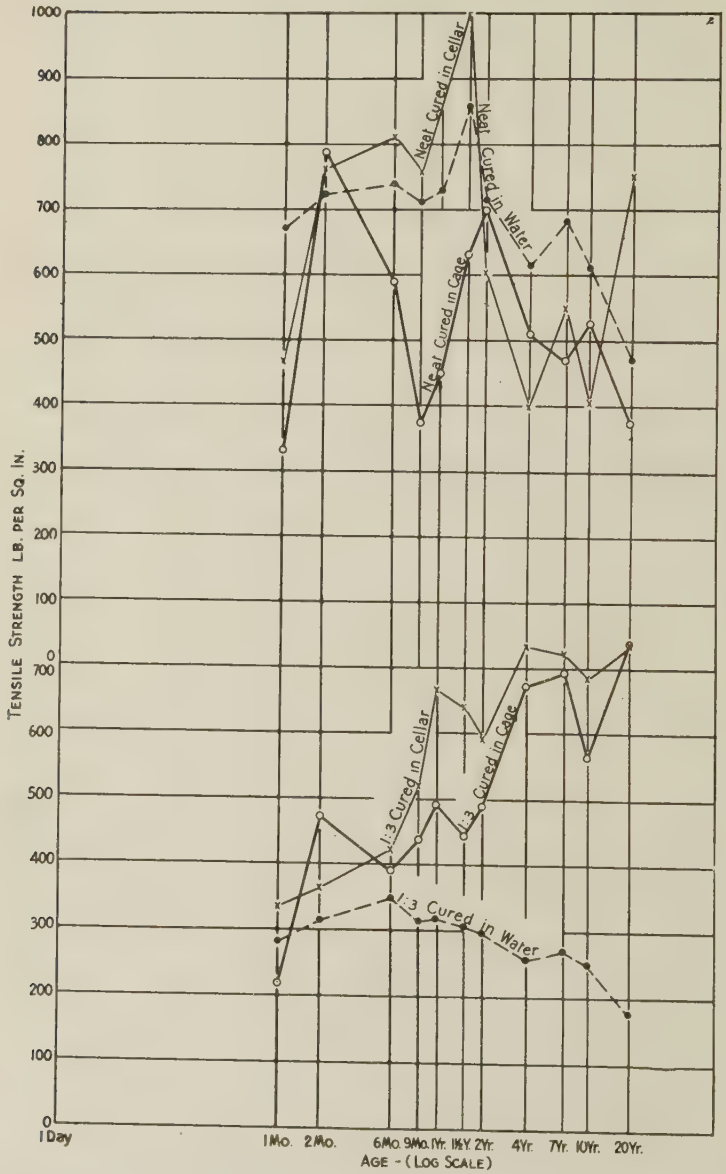


FIG. 2—RELATIONSHIP BETWEEN AGE AND CURING CONDITIONS ON THE TENSILE STRENGTH OF ATLAS CEMENT MORTAR BRIQUETS

in the cellar increased approximately as the logarithm of the age, as Fig. 2 shows, with the cellar-cured specimens showing slightly higher tenacity. On the other hand, the tensile strength of the water-cured 1:3 mortar specimens exhibited a decided retrogression after six months. Chemical analyses of the 1:3 briquettes subjected to the different storage conditions for 20 years showed the following cement contents in terms of the weight of the sand: for the cellar-cured specimens 34 per cent, for the cage-cured specimens 33 per cent, and for the water-cured specimens 31 per cent. From these data it seems likely that leaching out of the cement from the surfaces of the porous water-cured 1:3 briquets was the probable cause of their weakness in the late tests.

Effect of Curing Conditions on Compressive Strength of Concrete—Curves showing the relationship of the compressive strength of the concretes cured in the three ways to the logarithm of the age at test are shown in Fig. 3. Each point represents the average of five tests. It is apparent that the compressive strength of both the 1:2:4 and the 1:3:6 mixes cured out of doors or in the water tank increased approximately as the logarithm of the age. These two storage conditions gave approximately equal strengths. It is of interest to note that the 1:2:4 mixes cured under the more favorable conditions were slightly more than 2½ times as strong at 20 years as at one month, and that the 1:3:6 mixes under similar conditions were over 3½ times as strong. This increased percentage gain should not be interpreted as meaning that the weaker mixtures are gaining on the stronger, for Fig. 3 shows that the general trend of the curves is divergent and that the margin of strength between the rich and lean mixes is widening as the age increases. Both mixes of concrete cured in the cellar exhibited at 20 years about twice the one-month strength.

TESTS OF SERIES B

Materials

Cements—To ascertain the effect of different brands of cement on the age-strength relationships, three 10-bag shipments from each of four brands commonly used in Wisconsin were purchased from different consignments. Each shipment was sampled and tested and also the mixture of the three shipments of each brand. The four mixtures were used in the major tests of Series B. Table

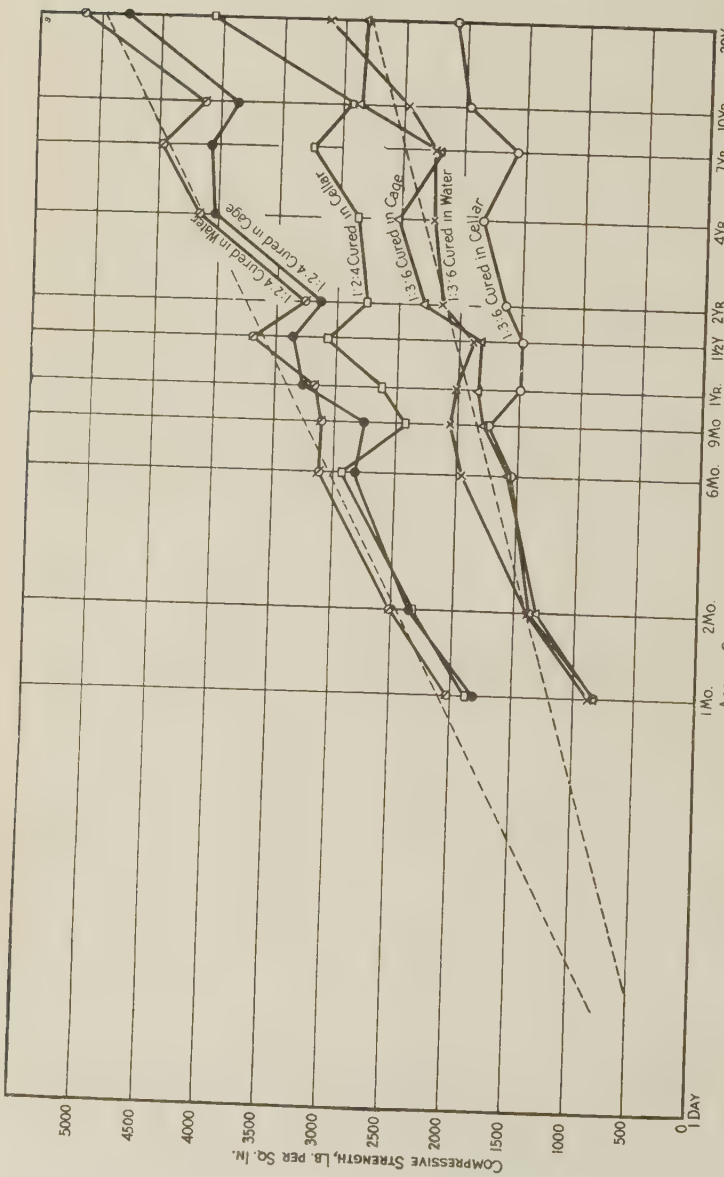


FIG. 3—STRENGTH-AGE CURVES FOR CONCRETE CURED IN VARIOUS MANNERS

1 shows results of standard physical tests on the mixtures representing the four brands. All brands exhibited good strength properties at 3, 7, and 28 days and passed other standard requirements. However, between the 90-day and 5-year periods all brands showed retrogression in both neat cement and mortar strengths. It is of interest to note that brand 3M which exhibited the greatest retrogression in mortar strength was the only brand which had a higher neat strength at five years than at one year.

TABLE 1—PHYSICAL PROPERTIES OF CEMENTS

Materials: Standard Ottawa sand. Cement samples taken from a mixture of 3 shipments.
All specimens were removed from molds at the age of one day and cured under water until tested.
Soundness in air, water, and steam, O. K.

Brand	Age		Tensile Strength, Lb. Per Sq. In.		Specific Gravity	Per Cent Water in Mix	Residue on Sieve No. 200	Time of Set					
								Initial				Final	
								Vicat		Gillmore		Vicat	Gillmore
	Years	Days	1:3 By Weight	Neat				Hr.	Min.	Hr.	Min.	Hr. Min.	Hr. Min.
Medusa (3M)		7	290	643	3.12	10.3	23.5	2	52	4	08	7 07*	7 53
		28	413	668									
		90	441	782									
	1	0	388	696									
	2	0	333	610									
	5	0	261	750									
Lehigh (4M)		7	279	608	3.14	10.2	21.7	2	12	3	32	5 25	6 52
		28	362	626									
		90	416	698									
	1	0	407	653									
	2	0	362	556									
	5	0	316	508									
Universal (5M)		7	264	667	3.13	10.3	20.7	1	55	3	12	5 00	6 28
		28	386	692									
		90	360	705									
	1	0	377	543									
	2	0	335	532									
	5	0	316	485									
Marquette (7M)		7	281	601	3.16	10.2	18.3	1	43	2	57	4 48	5 57
		28	394	643									
		90	427	672									
	1	0	437	592									
	2	0	390	514									
	5	0	372	486									

*Average of tests on individual shipments.

Chemical compositions of these cements are given in Table 2. The calculated constitutions of the cements are given in Table 9.

Autoclave tests were also made on the cements, but the results appear to have little correlation to the data thus far obtained in other tests.

TABLE 2—CHEMICAL ANALYSIS OF CEMENTS*

Brand	Specific Gravity	Chemical Analysis in Per Cent								
		SiO_2	CaO	MgO	Al_2O_3	Fe_2O_3	SO_3	Loss on Ignition	Total	Insoluble Residue
Medusa (3M)	3.12	22.53	61.40	4.39	5.43	2.56	1.73	2.32	100.36	0.22
Lehigh (4M)	3.14	22.21	61.16	2.57	7.32	2.69	1.90	1.64	99.49	0.44
Universal (5M)	3.13	21.41	62.55	2.30	7.18	2.57	1.53	2.40	99.94	0.16
Marquette (7M)	3.16	23.48	60.49	3.57	5.92	3.10	2.04	1.65	100.25	0.43

*Made by Professor O. A. Hougen.

TABLE 3—PROPERTIES OF AGGREGATES USED IN CONCRETE MIXES

Aggregate	Specific Weight, Lb. per cu. ft.	Specific Gravity	Per Cent Absorption	Fineness Modulus (m)	Sieve Analysis				
					Sieve No.	Per Cent Coarser Than Each Sieve			
						Janesville Sand	Janesville Gravel	Lannon Dolomite	Red Granite
Janesville Sand	112.6	2.70	0.60	2.86	1½	0	0	0	0
					¾	0	42.8	42.8	36.4
					⅜	0	85.7	85.7	90.9
Janesville Gravel	105.0	2.67	1.30	7.28	4	2.7	100.0	100.0	100.0
					8	17.5	100.0	100.0	100.0
					14	30.9	100.0	100.0	100.0
Lannon Dolomite	92.0	2.73	0.70	7.28	28	48.9	100.0	100.0	100.0
					48	88.1	100.0	100.0	100.0
					100	98.3	100.0	100.0	100.0
Red Granite	94.0	2.62	0.19	7.27		m = 2.86	m = 7.28	m = 7.28	m = 7.27

Janesville gravel contained approximately 50 per cent broken fragments, and is composed of approximately ¾ dolomite, 1/5 quartz, the remainder being igneous material.

Janesville sand contained approximately 3/5 quartz, ¼ dolomite, and the remainder was largely igneous material.

Red granite is composed essentially of quartsite and feldspar.

Aggregates—Table 3 shows the physical properties of the aggregates used in tests of Series B. The Janesville sand and gravel are representative of the better types of glacial sand and gravel commonly used in concrete construction in Southern Wisconsin. Lannon dolomite and red granite represent hard and durable crushed rock aggregates of their respective types. The gravel particles were the most nearly equiaxed, the granite particles the least. It will be noted that all the coarse aggregates had approximately the same grading as indicated by the values of the fineness modulus (m).

Making and Testing Specimens

Auxiliary Test Specimens of Mortars—Besides the standard strength tests of cements, additional tensile and compressive tests of mortars made with standard sand and others made with Janesville sand are scheduled as outlined in Table 4.

TABLE 4—SCHEDULE OF MORTAR TESTS OF SERIES B

Brand of Cement	Kind of Sand	Mix by Weight	Curing Condition	Age at Test	Number of Specimens to be Tested			
					Per Mix Per Period		Total Made	
					2 x 4-in. Cylinders	Briquettes	2 x 4-in. Cylinders	Briquettes
3M 4M 5M 7M	—	Neat	Cage after 28 days in water	7 da. 28 da. 90 da. 1 yr. 2 yr. 5 yr. 10 yr. 25 yr. 50 yr. 100 yr.	3	3	120	120
As above	Standard	1:1 1:2 1:3	As above	As above	3	3	360	360
As above	Janesville	1:1 1:2 1:3	As above	As above	3	3	360	360
As above	Standard	1:3	In water	As above	3		120	
					Total. .	. . . . .	960	840

The consistencies for these mortar tests were based on Feret's formula,

$$w = \frac{2}{3}\frac{p}{s+1} + k;$$

where w = percentage of water for the given mix, s = number of parts sand to one of cement, by weight, p = percentage of water for standard neat consistency, and k = 6.5 for standard sand and 6.9 for Janesville sand.

Mortar briquets and cylinders were made in groups of six in separate batches. Specimens were numbered and stored so that not more than one specimen of a given batch was tested at an age. Specimens were cured one day in molds, 27 days in water. Those to be stored outdoors were then placed on wood shelves in the same cage used for the storage of specimens of Series A.

Concrete Test Specimens—Cylinders 6 in. in diameter and 12 in. high were made in batches of 20 in accordance with the program in Table 5. The materials were weighed and mixed in the same type of mixer as specimens of Series A. Dry cement and aggregate were mixed for ½ min., then the water was added and mixing was continued for 1½ min. Each batch from the mixer was discharged into wheelbarrows and thence onto a mixing tray where it was turned over to render it homogeneous. The requisite water for the desired consistency was determined by hand-mixing small preliminary batches. The consistency of every batch was checked on a 30-in. flow table using an 8 x 12 x 6-in. frustrum and 15 ½-in. drops in 15 sec. Molds were filled quarter full at a time and each layer was puddled 25 times with a ½-in. rod. A neat cement cap was placed on each cylinder about four hours after molding. After one day in the molds specimens were marked and stored for 27 days in a moist closet. Then they were removed and subjected to the appropriate storage conditions. The outside curing is conducted in the same cage as the specimens of Series A. The inside curing is done in a room where the temperature ranges from 65 to 85 deg. F. and the humidity ranges from 40 to 80 per cent. The schedule is so arranged that not more than two cylinders from a given batch are broken at a given age.

The 4 x 6 x 24-in. contraction prisms were made from certain of the batches used in making cylinders. Brass plugs spaced

TABLE 5—SCHEDULE OF CONCRETE TESTS OF SERIES B

Janesville sand was used in making all specimens.
Specimens for compression, absorption, and freezing tests were 6 x 12-in. cylinders; for contraction tests were 4 x 6 x 24-in. prisms.

Type of Test	Brand of Cement	Mix by Vol.	Coarse Aggregate	Consistency*	Flow	Curing Conditions	Age at Test	Number of Specimens to be Tested			
								Per Mix Per Period		Total Made	
								Cyl.	Prisms	Cyl.	Prisms
Compression and Expansion	3M 4M 5M 7M	1:2:4	Janesville Gl. Lannon Dol. Red Granite	110	165	Cage	7 da. 28 da. 90 da. 1 yr. 2 yr. 5 yr. 10 yr. 25 yr. 50 yr. 100 yr.	5	3	600	36
Compression and Contraction	As above	1:1½:3	Janesville Gravel	110	165	Cage	As above	5	3	200	12
		1:3:6		110	165	Cage		5	3	200	12
		1:2:4		140	250	Cage		5	3	200	12
		1:2:4		110	165	Indoors		5	3	200	12
		1:2:4		140	250	Indoors		5	3	200	12
Absorption and Freezing	As above	1:2:4	Janesville Gl.	110	165	Moist clos.	28 da.	10		40	
		1:2:4	Lannon Dol.	110	165	Moist clos.	28 da.	10		40	
		1:2:4	Red Granite	110	165	Moist clos.	28 da.	10		40	
		1:1½:3	Janesville Gl.	110	165	Moist clos.	28 da.	10		40	
		1:3:6	Janesville Gl.	110	165	Moist clos.	28 da.	10		40	
		1:2:4	Janesville Gl.	140	250	Moist clos.	28 da.	10		40	
										1840	96

*Percentage of water used in mix based on net amount required for a slump of one inch.

TABLE 6—AVERAGE CEMENT CONTENTS, WATER CEMENT RATIOS AND DENSITIES OF MIXES

Mix By		Coarse Aggregate	Cement Content Sacks Per Cu. Yd.	Water Cement Ratio, w/c	Flow in Per Cent	Specific Weight, Lb. Per Cu. Ft.	Density p
Vol.	Weight						
1:2:4	1:2.4:4.47	Gravel	5.15	0.76	165	151.5	0.828
1:2:4	1:2.4:4.47	Gravel	5.04	1.01	250	150.7	0.808
1:2:4	1:2.4:3.92	Lannon Dol.	5.52	0.81	165	151.6	0.814
1:2:4	1:2.4:4.00	Red Gr.	5.41	0.77	165	149.1	0.824
1:3:6	1:3.6:6.70	Gravel	3.58	1.04	165	150.5	0.830
1:1½:3	1:1.8:3.35	Gravel	6.62	0.61	165	152.2	0.828

20 in. apart were molded in the axis of each 6 in. face. These were provided with holes for strain gage readings. The slabs were subjected to the same curing conditions as the cylinders.

The 6 x 12-in. cylinders for the absorption and freezing tests were selected from those made in different batches. The absorption cylinders were roughened with wire brushes when one day old. For both types of test the cylinders were cured in the moist closet for 27 days.

Methods of Testing

Absorption Tests—After the roughened 6 x 12-in. cylinders for the absorption tests had been stored 27 days in the moist closet they were dried to constant weight at temperatures just above the boiling point of water. They were then immersed in water at room temperature for 48 hr. to determine the percentage of absorption. After an interval of a few days the cylinders were boiled in water for 5 hr. to ascertain how much additional water they would absorb. The percentages of absorption were based on weights.

Method for Freezing and Thawing Tests—The cylinders to be subjected to the freezing and thawing tests, after curing in moist closet for 28 days, were immersed in hot water, at approximately 190 deg. F., for several hours to saturate them. They were cooled in running tap water and then placed in a freezing chamber having an average temperature of 21 deg. F., and kept there until thoroughly frozen. All of these specimens were subjected to twenty such cycles. They were then broken in compression.

Contraction Measurements—Measurements on the lengths of the 4 x 6 x 24-in. contraction prisms were made with a 20-in. Berry strain gage reading to 0.0001 in. Initial readings were taken when the specimens were one day old. They were again read after 27 days in the moist closet while still in the closet and then removed to the assigned storage condition. Measurements of lengths and weights have since been taken at regular intervals, usually at the test periods for the concrete cylinders.

Procedure in Strength Tests—Mortar and concrete specimens stored in the cage have been removed to the laboratory and stored two days prior to testing. Other specimens were carried directly from storage to testing machine. Wherever necessary

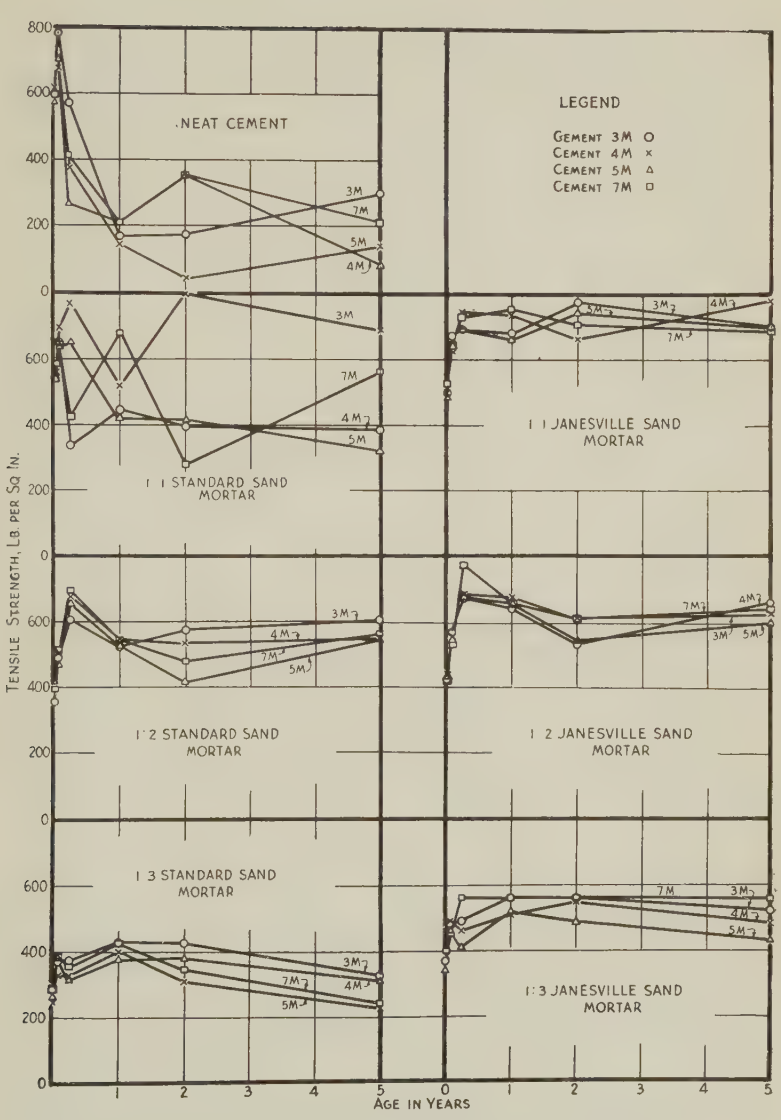


FIG. 4—RESULTS OF TENSION TESTS ON MORTAR AND NEAT CEMENT BRIQUETS CURED OUTSIDE AFTER 28 DAYS

defective caps have been removed from compression cylinders and they have been recapped with plaster of Paris.

The 6 x 12-in. concrete compression cylinders have been broken on a 200,000-lb. 3-screw Riehlé testing machine equipped with a spherical bearing on the movable head. The speed used in applying maximum loads was 0.06 in. per min.

The 2 x 4-in. mortar cylinders have been broken at a speed of 0.03 in. per min. under a small spherical bearing.

Briquettes have been tested in a Riehlé automatic shot briquet tester equipped with standard roller bearings.

RESULTS OF TESTS OF SERIES B

Mortar Strength Tests—Fig. 4 shows the effect of outside curing on the tensile strength of briquets made in 1923. These data show that neat cement briquets and 1:1 standard sand mortar briquets suffered severely from weathering. In some instances briquets of these rich mixes were badly checked and gave little strength. Since these very low values were averaged with the other results, they account for the marked fluctuations in the tensile-strength-age curves. Why the tenacity of the 1:1 mortars of standard sand suffered so markedly in contrast to 1:1 mortars of Janesville sand is not apparent. All 1:3 standard-sand mortars cured outside exhibited a pronounced retrogression in tensile strength after the one-year period. The 1:3 Janesville-sand mortars also showed a slight retrogression after the one-year period. The 1:2 mortars and the 1:1 Janesville-sand mortars showed little change in tensile strength after the one-year period.

The results of the compression tests (not presented herein) on 2 x 4-in. mortar cylinders cured outside show much less variation in compressive strength than in tensile strength. Furthermore, there is no pronounced retrogression in strength with age for any of the mixes. As in the tensile tests, the Janesville sand mortars exhibited compressive strengths superior to those attained by standard sand mortars, especially in the 1:3 mixes.

In the mortar tests the influence of brand on strength was not large. Considering the results of tests with Janesville sand and averaging the data for the 28-day to 5-year periods, brand 7M showed the highest tensile strength and brand 5M the lowest. The difference between the average for the highest brand and the

lowest brand amounted to 4 per cent for the 1:1, 7 per cent for the 1:2, and 15 per cent for the 1:3 mix. In compression brand 4M gave the highest average strength and brand 5M the lowest. The difference between the average for the highest and lowest brands was 9 per cent for the 1:1 mix, 8 per cent for the 1:2 mix, and 6 per cent for the 1:3 mix.

The differences in tensile strengths of the neat cement mixes due to curing conditions was very marked. Neat briquettes stored in water exhibited much higher and more uniform strengths than those stored outside. The discrepancies in the tensile strengths of the 1:3 standard sand mortar briquets due to the two storage conditions was not marked, but the 1:3 cylinders of standard sand cured outside averaged about fifty to seventy-five per cent stronger than companion specimens cured in water at room temperature.

It is believed that the wide discrepancies in the tensile strengths of the specimens cured outside was due to surface checks which seriously diminish tenacity, especially of small specimens, but which affect the compressive strength little.

Physical Properties of Concretes—Data on average cement contents, water-cement ratios, consistencies and densities of the concretes used in these tests are given in Table 6. The water-cement ratios are based on ratio of the volume of free water (absorbed water subtracted) to the volume of cement. Cement contents, specific weights, and densities were determined on concrete after removal from molds.

Results of Absorption Tests—Data on the absorption tests are given in Table 7 and in Fig. 5. The absorptions of the individual cylinders were very uniform in most cases, the average variation from the mean in a group of five was less than 5 per cent and in only one case did it reach 8 per cent. The data in column 7 of Table 7 show the variations of the mean for each brand from the average for the given mix.

The absorption for the 1:2:4 Red granite mix was least, and that mix also had the lowest ratio of absorption after 4-hr. immersion to absorption after boiling. Considering tests with Janesville gravel, the absorption appears to increase as the water-

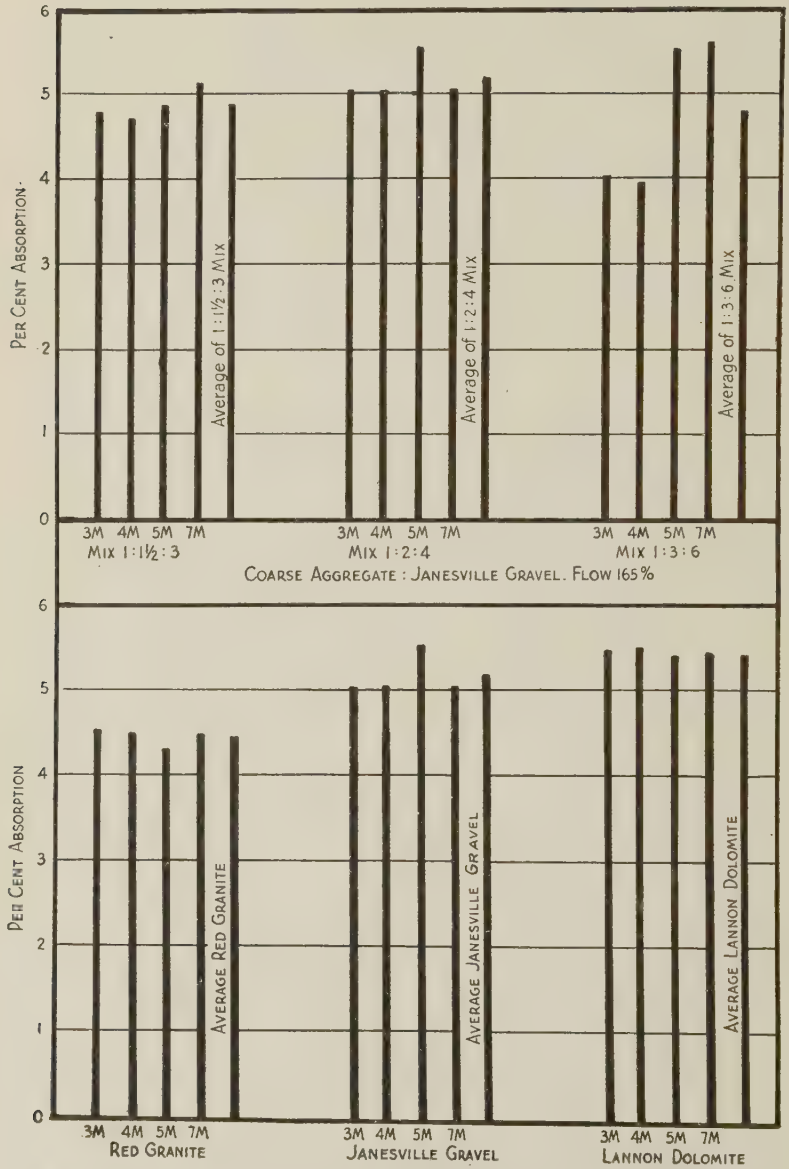


FIG. 5—EFFECT OF MIX AND TYPE OF AGGREGATE ON ABSORPTION OF CONCRETE

TABLE 7—RESULTS OF ABSORPTION TESTS ON CONCRETE

Specimens were stored in moist closet for 27 days, dried to constant weight, then immersed in water for 48 hours. After a few days the cylinders were boiled in water for half a day to show how much more additional water would be absorbed by this treatment.

Each value represents the average of 5 test specimens.

Brand of Cement	Mix	Flow, Per Cent	W/C Ratio	Aggregate	Per Cent Absorption at 48 Hr. (A)	Per Cent Variation from Ave.	Per Cent Increase in Weight Due to Boiling 5 Hours in Water After Drying (B)	Ratio, A/B
3M	1:2:4	165	.81	Lannon	5.51	+ 0.7	6.36	.87
4M			.81	Dolomite	5.47	0.0	5.84	.93
5M			.81		5.44	— 0.6	6.06	.90
7M			.81		5.47	0.0	6.20	.88
Ave.					5.47		6.11	.89
3M	1:2:4	165	.77	Red	4.59	+ 2.2	5.44	.84
4M			.77	Granite	4.53	+ 0.9	5.49	.82
5M			.77		4.34	— 3.3	5.19	.84
7M			.77		4.52	+ 0.7	5.37	.84
Ave.					4.49		5.37	.84
3M	1:2:4	165	.76	Janesville	5.02	— 2.9	6.14	.82
4M			.76	Gravel	5.03[a]	— 2.2	5.62[a]	.90[a]
5M			.76		5.58	+ 7.9	6.32	.88
7M			.76		5.04	— 2.5	5.77	.87
Ave.					5.17		5.96	.87
3M	1:2:4	250	1.01	Janesville	5.85	— 3.8	6.58	.89
4M			1.01	Gravel	6.30	+ 3.6	7.02	.90
5M			1.01		6.55	+ 7.7	7.16	.91
7M			1.01		5.64	— 7.2	6.64	.85
Ave.					6.08		6.85	.89
3M	1:1½:3	165	.61	Janesville	4.78	— 1.8	6.01	.80
4M			.61	Gravel	4.71	— 3.3	5.71	.82
5M			.61		4.83	— 0.8	5.68	.85
7M			.61		5.17	+ 6.2	5.73	.90
Ave.					4.87		5.78	.84
3M	1:3:6	165	1.04	Janesville	4.02	—16.1	4.66	.86[a]
4M			1.04	Gravel	3.95	—17.5	4.31	.92[b]
5M			1.04		5.55	+15.9	6.19	.90
7M			1.04		5.65	+18.0	6.35	.90
Ave.					4.79		5.38	.90

[a]This value represents the average of 4 test specimens.
[b]This value represents the average of 2 test specimens

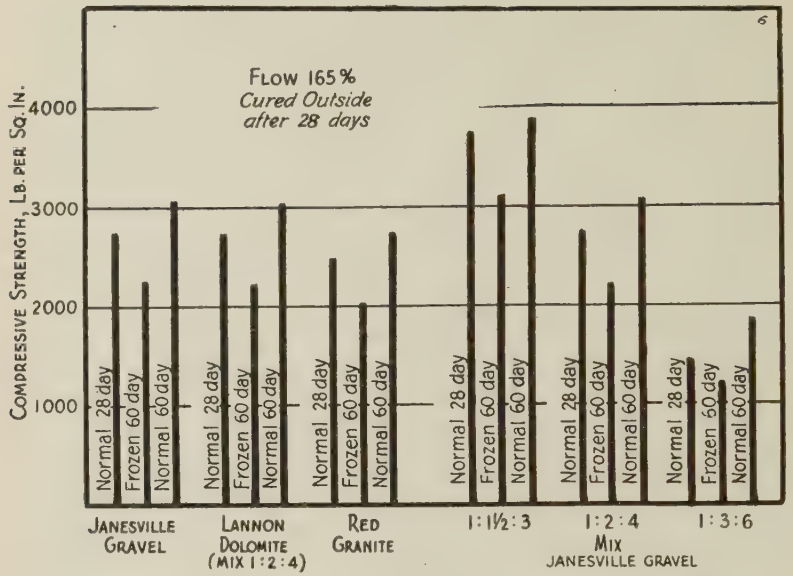


FIG. 6—EFFECT OF AGGREGATES AND PROPORTIONS ON STRENGTH OF CONCRETE SUBJECTED TO FREEZING AND THAWING

cement ratio (w/c) increases. A higher w/c undoubtedly accounts for the absorption of the 1:2:4 Lannon stone mix being greater than the absorption of the 1:2:4 mix of Janesville gravel.

Neglecting values for the 1:3:6 mix, in which the values for brands 3M and 4M appear abnormal, the average absorptions for 25 specimens of a given brand were:

Brand	3M	4M	5M	7M
Average Percentage Absorption	5.15	5.21	5.35	5.17

Hence there appears to be little difference in the absorption of concrete made from these brands of cement.

Results of Freezing and Thawing Tests—The data from the freezing and thawing tests on the concrete cylinders are summarized in Table 8 and Fig. 6. Although the freezing and thawing tests were started when the specimens were 28 days old the

TABLE 8—RESULTS OF FREEZING TESTS

Test specimens were cured normally for 28 days, then subjected to 20 cycles of freezing at a temperature of approximately 21° F. and thawing in water heated at a temperature of approximately 190° F. Temperature reversals extended over a period of one month.

All normal specimens were cured outside after 28 days.

The 60-day normal strength values were interpolated from strength curves plotted against the logarithm of the age.

A/B is the ratio of the percentage absorption at 48 hours to the percentage increase in weight due to boiling 5 hours in water after drying.

Mix	Coarse Aggregate	Cement Brand	W/C Ratio	Flow, Per Cent	Comp. Strength, Lb. per Sq. In.			% Reduction in Str. of Frozen Concrete Based on 60-Day Normal Str.	Ratio, A/B
					Concrete Subjected to Freezing, Age 60 Days	Normal Concrete, Age 90 Days	Normal Concrete Corresponding to Age of Frozen		
1:2:4	Granite	3M	.77	165	1960	2465	2285	14	.87
		4M	.77	165	1995	3065	2845	30	.93
		5M	.77	165	2060	2985	2890	29	.90
		7M	.77	165	2265	3155	3075	26	.88
Ave.					2070	2915	2775	25	.89
1:2:4	Lannon Dolomite	3M	.81	165	2150	2820	2705	21	.84
		4M	.81	165	2565	3260	3085	17	.82
		5M	.81	165	1995	3310	3050	34	.84
		7M	.81	165	2140	3400	3360	36	.84
Ave.					2215	3200	3050	27	.84
1:2:4	Janesville Gravel	3M	.76	165	2250	3110	2875	22	.82
		4M	.76	165	2285	3695	3435	33	.90
		5M	.76	165	1860	2785	2840	34	.88
		7M	.76	165	2640	3325	3145	16	.87
Ave.					2260	3230	3075	27	.90
1:2:4	Janesville Gravel	3M	1.01	250	1255	1985	1800	30	.89
		4M	1.01	250	1535	2470	2230	31	.90
		5M	1.01	250	1345	2130	2130	37	.91
		7M	1.01	250	1655	2440	2220	25	.85
Ave.					1445	2255	2095	31	.89
1:1½:3	Janesville Gravel	3M	.61	165	2795	3690	3560	21	.80
		4M	.61	165	3300	3635	3575	8	.82
		5M	.61	165	3275	4230	4165	21	.85
		7M	.61	165	3365	4480	4345	23	.90
Ave.					3185	4010	3910	19	.84
1:3:6	Janesville Gravel	3M	1.04	165	1100	1610	1520	28	.86
		4M	1.04	165	1140	2235	1975	42	.92
		5M	1.04	165	1400	2105	1880	25	.90
		7M	1.04	165	1285	2205	1980	35	.90
Ave.					1230	2040	1835	33	.90

20 cycles were not completed until the specimens were about 60 days old. Since no tests were made on normal concrete at 60 days, for purposes of comparison the strengths of normal specimens in column 8 of Table 8 were estimated by interpolating between the 28-day strengths and the 90-day strengths. In making this interpolation the strength was assumed to vary with the logarithm of the age.

It will be noted from the data in column 9, Table 8 and Fig. 6 that in every case the concrete which was subjected to freezing and thawing had less strength than the normally-cured concrete 28-days old and considerably less strength than the estimated 60-day strength for normal concrete. Considering all mixes, the concrete which was frozen and thawed had an average strength eighteen per cent less than the normally-cured concrete 28-days old and 26 per cent less than the estimated strength of normally-cured concrete 60-days old.

From the chemical analyses in Table 2 the constitution of each of these cements has been calculated and the percentages of the components are listed in Table 9. In the same table are listed the average absorption data from Table 7, the values of Schurecht's ratio, A/B, from column 10 of Table 8, and the average percentage loss in compressive strength due to freezing. Values for the 1:3:6 mix are excluded in these tabulations. Comparisons show that there was not a great difference in the freezing resistances of the concretes made of these different cements, that made from cement 3M had the lowest absorption, the lowest Schurecht's ratio (A/B), and the lowest loss in strength due to freezing and thawing.

A study of these data indicates that the losses in the freezing and thawing test were greatest for mixes having high water ratios and high absorptions and least for those mixes made with lowest water ratios and having the lowest absorptions. Hence, as Fig. 6 shows, the 1:1½:3 mix of high cement content and low water-cement ratio exhibited the lowest loss, 19 per cent, in freezing and thawing, whereas the lean 1:3:6 mix showed the highest loss, 33 per cent. Making allowances for the discrepancies in the water-cement ratios of the 1:2:4 mixes made with different aggregates, a comparison of the data in Fig. 6 indicates that there

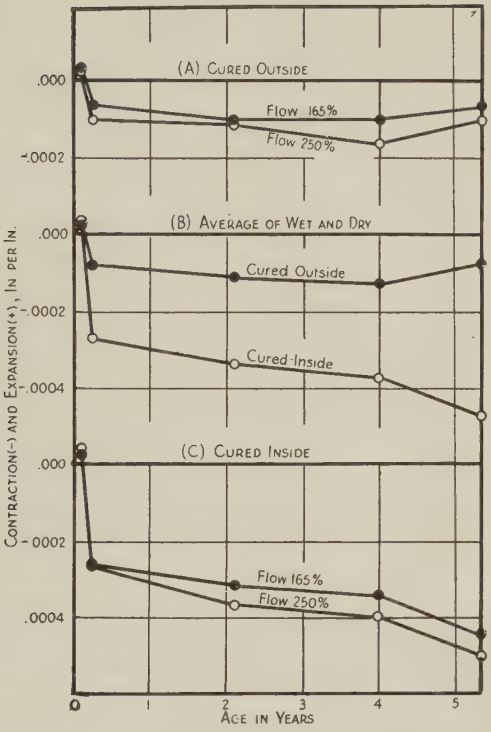


Fig. 7—Expansion and contraction of 1:2:4 concrete of different consistencies made of Janesville gravel

was little difference in resistance to freezing of the concretes made with these aggregates.

Results of Expansion and Contraction Tests—The data on effect of outside curing on the linear contraction of 4 x 6 x 24-in. slabs show that at 1 month the mean linear dimensional change in all cases was a small initial expansion (0.00006 to 0.00008) due to curing in the moist closet, but at subsequent periods the outside curing caused a somewhat greater contraction in nearly all cases. The data indicate that the average contraction on concrete made with cement 3M was the least of the average values for the four cements used in these tests. Data from the concrete cured indoors indicated a similar relationship.

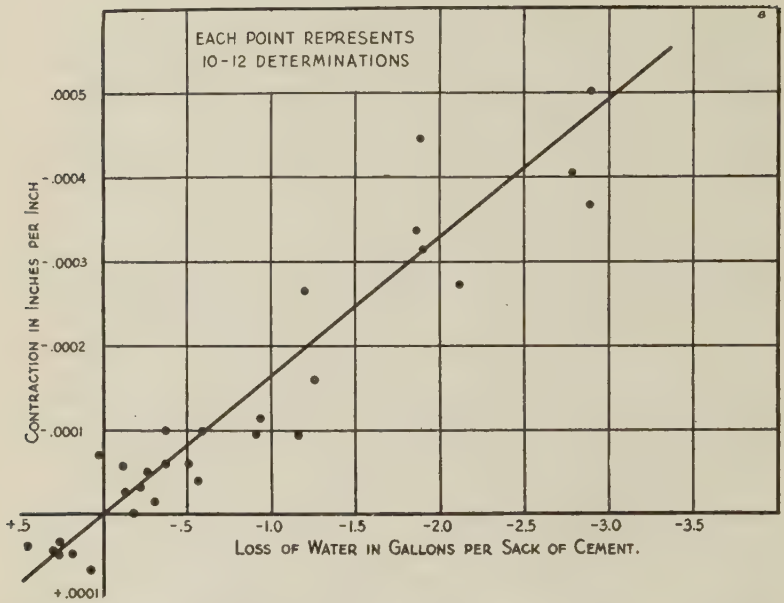


FIG. 8—THE RELATIONSHIP BETWEEN EXPANSION, CONTRACTION, AND LOSS OF WATER IN CURING FOR 1:2:4 CONCRETE

Cement Content of Concrete 5.0 to 5.5 Sacks per cu. yd.

Concrete made of the dolomite showed the most expansion and the least contraction due to outside curing, whereas that made of gravel showed the greatest contraction. In no cases, however, did the average change exceed 0.0001.

The evidence concerning the effect of proportion of cement, is not conclusive, but it indicates that the contraction in curing outside was greatest for the 1:1½:3 mix.

The curves in Fig. 7 (A) and (C) show that the 1:2:4 concrete with a flow of 165 per cent contracted less than the wetter mix with a flow of 250 per cent. Fig. 7 (B) indicates that curing inside caused three to six times as much contraction as the outside curing. At five years contraction of the 1:2:4 concrete due to indoor curing amounted to .04 to .05 per cent.

The most significant relationship obtained in these expansion and contraction tests is that between the percentage of water lost

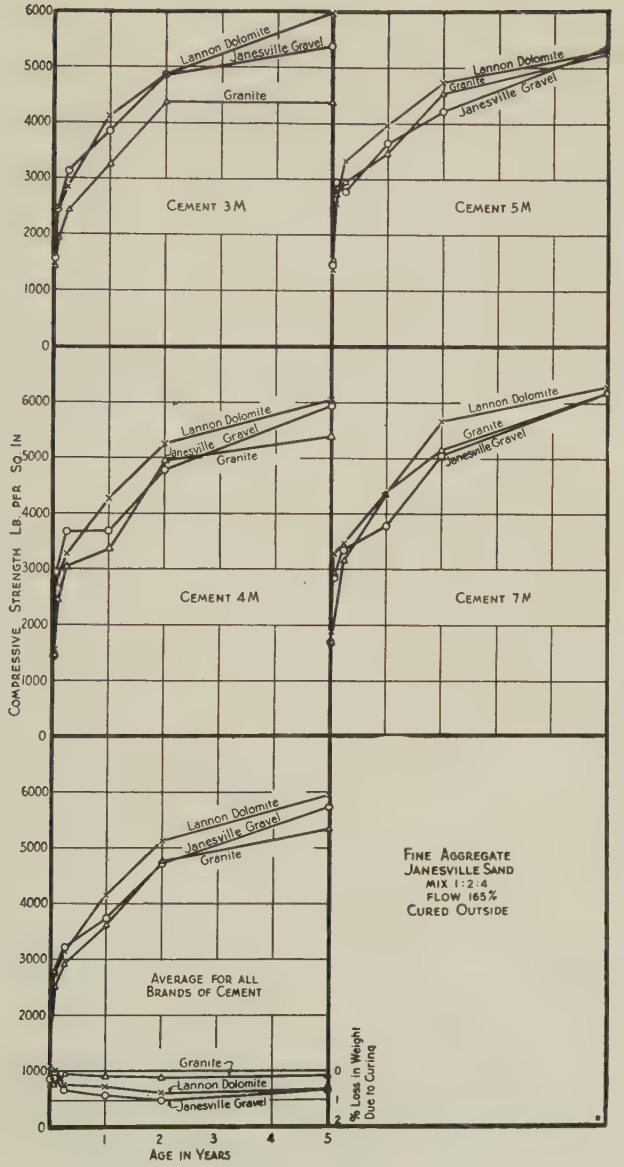


FIG. 9—STRENGTH-AGE RELATIONSHIP FOR CONCRETE MADE OF DIFFERENT BRANDS OF CEMENT AND DIFFERENT AGGREGATES

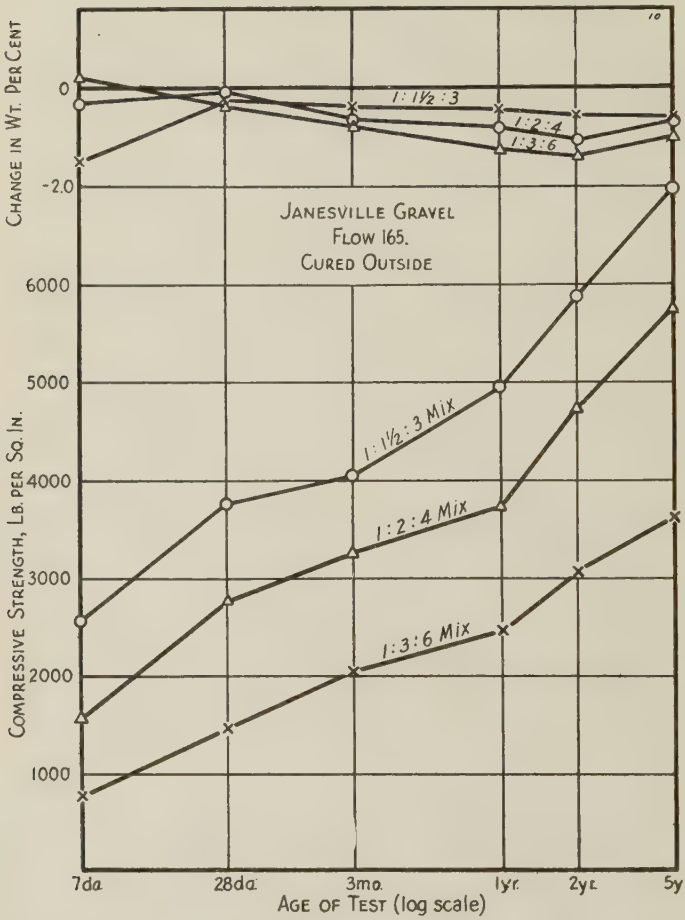


FIG. 10—EFFECT OF MIX ON THE COMPRESSIVE STRENGTH OF CONCRETE AND ON LOSS IN WEIGHT DUE TO CURING

and the change in linear dimension, see Fig. 8. Doubtless most of the discrepancies are due to losses in weight from spalling of the specimens in handling. The line in Fig. 8 checks very closely data obtained at the Portland Cement Association Research Laboratory by M. B. Lagaard.* With this relationship in mind,

*See diagram in Johnson's *Materials of Construction*, 7th Ed., p. 480

one would expect that the contraction would be least for the cement paste exhibiting the lowest absorption. The data herein, though meager, substantiate this view.

Results of Compression Tests of Concrete—The effects of age on the compressive strength of 1:2:4 concrete cured outside are plotted in Fig. 9 to a uniform age scale. The strength-age data for the tests on 1:1½:3, 1:2:4 and 1:3:6 mixes of Janesville sand and gravel are plotted to a logarithmic age scale in Fig. 10. No retrogressions in strength are exhibited in any of these curves, but the retarding influence of seasonal variations in temperature and humidity during the first year are manifested by the corresponding irregularities in the strength-age curves. As in tests of Series A, the data of Fig. 10 show that the strength increased with the logarithm of the age at test.

The curves in Fig. 9 show that the concrete made of Lannon dolomite was slightly stronger than that made of the other aggregates despite the fact that it was mixed with a higher water cement ratio. The concrete from this aggregate also contained a larger proportion of cement than the concrete made with the gravel. The latter fact and the superior bond afforded by the surfaces of the angular dolomite particles undoubtedly account for the superior strength of the dolomite concrete.

The effects of the outside and of the inside curing conditions on the age-strength relationships are shown in Fig. 11. Concrete cured inside exhibited little change in strength after the 3 month tests, and at 5 years had only about 65 per cent of the strength of concrete cured outside.

The data show that 1:2:4 concrete with a flow of 165 percent (w/c = 0.76) had, when cured outside for one year, 35 per cent greater strength than similar concrete with a flow of 250 (w/c = 1.01). At 5 years the mix with 165 per cent flow had 50 per cent greater strength than the wetter mix. Under inside curing the drier mix exhibited from 25 to 35 per cent greater strength than the mix with a flow of 250 per cent.

Since loss in weight during curing is a measure of the desiccating influence of the storage medium on the concrete, it is of interest

TABLE 9—A COMPARISON OF CALCULATED CONSTITUTIONS OF CEMENTS 3M, 4M, 5M, AND 7M WITH RESULTS OF VARIOUS TESTS

Brand of Cement	5M	3M	4M	7M
Per cent $3CaO.SiO_2$	35.1	33.5	21.3	17.6
Per cent $2CaO.SiO_2$	35.0	39.4	47.7	54.2
Per cent $3CaO.Al_2O_3$	14.7	10.1	14.9	10.4
Per cent $4CaO.Al_2O_3.Fe_2O_3$	7.8	7.7	8.2	9.4
Per cent $CaSO_4$	2.6	2.9	3.2	3.5
Per cent MgO	2.3	4.4	2.6	3.6
Ignition loss	2.4	2.3	1.6	1.6
Tensile strength 1:3 Std. Sd. mortar, 28 days, lb/in.^{2}	330	370	386	372
Tensile strength 1:3 Std. Sd. mortar, 5 yr., lb/in.^{2}	233	326	319	246
Tensile strength 1:2 J. Sd. mortar, 28 days, lb/in.^{2}	533	561	575	557
Tensile strength 1:2 J. Sd. mortar, 5 yr., lb/in.^{2}	607	632	664	645
Compressive strength 1:3 Std. Sd. mortar, 28 days, lb/in.^{2}	2750	2490	2720	2450
Compressive strength 1:3 Std. Sd. mortar, 5 yr., lb/in.^{2}	4770	5610	5790	5540
Compressive strength 1:2 J. Sd. mortar, 28 days, lb/in.^{2}	5180	4890	5870	5790
Compressive strength 1:2 J. Sd. mortar, 5 yr., lb/in.^{2}	6950	8050	8520	7690
Gain in neat strength in autoclave, per cent	48	25	24	34
Expansion in autoclave, per cent	.09	.21	.13	.14
Ave. percentage absorption of 1:2:4 concrete	5.35	5.15	5.21	5.17
Ave. Schurecht's ratio	0.88	0.84	0.87	0.87
Ave. loss of crushing strength of concrete due to freezing and thawing, per cent	31	22	24	25
Ave. contraction of 1:2:4 concrete, flow 165 per cent cured outside 4 yr., per cent	0.010	0.0007	0.007	0.008
Ave. contraction of 1:2:4 concrete, flow 165 per cent cured inside 4 yr., per cent	0.036	0.027	0.032	0.040
Ave. crushing strength of 1:2:4 concrete, flow 165 per cent, cured in moist closet 1 mo., lb/in.^{2}	2945	2430	2950	2800
Ave. crushing strength of 1:2:4 concrete, flow 165 per cent, cured outside 5 yr., lb/in.^{2}	5395	5370	5925	6190
Ave. crushing strength of 1:2:4 concrete, flow 250 per cent, cured in moist closet 1 mo., lb/in.^{2}	2125	1445	1780	1790
Ave. crushing strength of 1:2:4 concrete, flow 250 per cent, cured outside 5 yr., lb/in.^{2}	3640	3505	3815	4280

to note the marked discrepancy between the percentage loss in weight of the concrete with flow of 165 cured indoors and the loss in weight of like concrete cured outside. At five years the concrete cured indoors had lost 3 per cent of its weight whereas that cured outside had lost about 0.7 per cent. Losses in weight sustained by the 1:2:4 concrete with a flow of 250 per cent at the

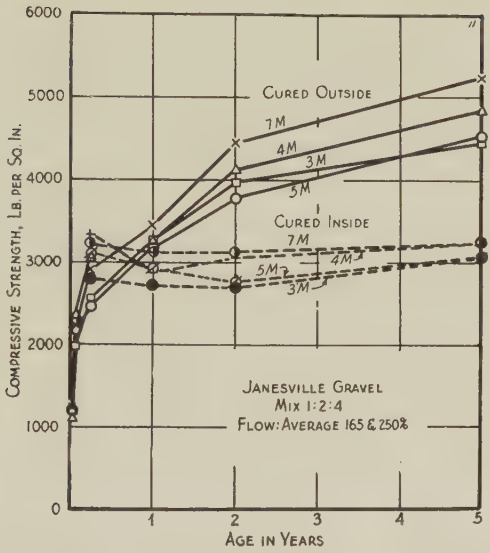


Fig. 11—Effect of curing conditions on strength of 1:2:4 concrete with different cements

same age were 3.6 per cent for the concrete cured inside and 1.1 per cent for that cured outside.

The linear relationship between the compressive strength of concrete of 165 per cent flow and the cement-voids ratio is well shown by the curves of Fig. 12. Each point in the diagram represents five tests.

The relationship of compressive strength of concrete to the water-cement ratio, by volume, is indicated in Fig. 13. Each point on these diagrams represents 20 tests.

Influence of Kind of Cement on Properties—Table 9 shows the amounts of the principal constituents in the different cements used in these tests. The percentage of these constituents were calculated from the chemical analysis by Bogue's method.* In Fig. 14 are shown the compressive strength-age curves for the concretes made of each brand of cement. In constructing these

*Paper No. 21, Portland Cement Association Fellowship at the National Bureau of Standards, Washington, D. C.

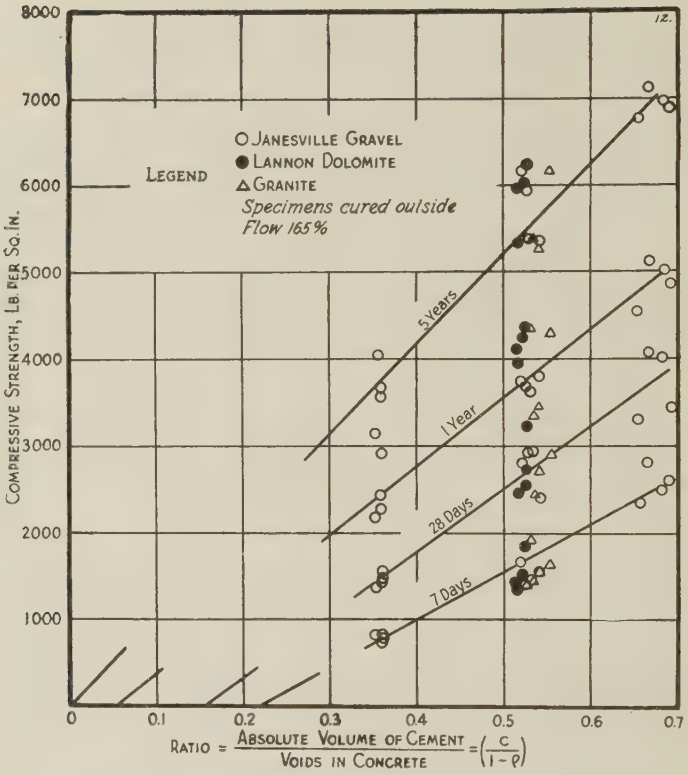


FIG. 12—RELATION BETWEEN THE COMPRESSIVE STRENGTH AND THE RATIO OF THE VOLUME OF CEMENT TO THE VOLUME OF VOIDS IN CONCRETE

curves the strengths of concretes of 165 per cent flow made with the various aggregates and proportions were averaged.

From the data in Table 9 it appears that there were considerable differences in the constitutions of these cements but the discrepancies in the various physical properties are much less. In most cases it will be noted that the cements containing the lower proportions of tri-calcium silicate (3 $CaO.SiO_2$) and the higher proportions of di-calcium silicate (2 $CaO.SiO_2$), brands 7M and 4M, exhibited in these long time tests slightly better strengths

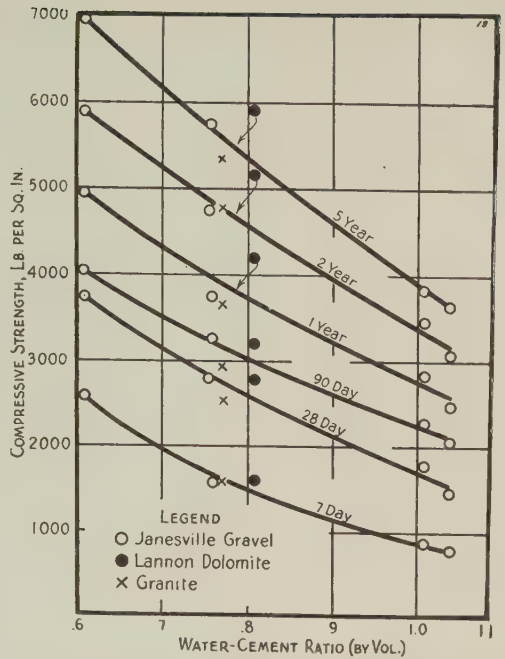


Fig. 13—Strength water-cement ratio relationship for concrete of different ages

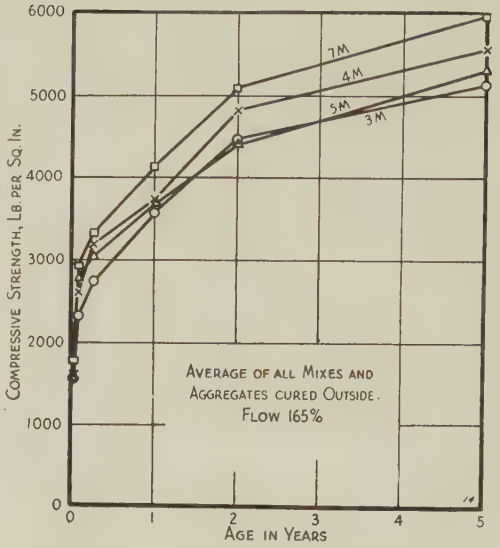


Fig. 14—Effect of brand of cement on the strength-age relationship

than the other two brands 5M and 3M which contained higher percentages of tri-calcium silicate.

As previously mentioned, the absorption and the loss in crushing strength due to freezing were slightly greater for the concrete of cement 5M (which was high in tri-calcium silicate and tri-calcium aluminate) than for the concretes made of other cements. The differences in other physical properties listed in Table 9 do not appear to be significant.

Comparison of Compressive Strengths of Concretes of Series A and B with Present Day Concrete—Table 10 has been inserted to furnish comparisons between the average compressive strengths of concrete of Series A (Fig. 3) and Series B (Fig. 13), cured outside, with similar data from the w/c-curves for the moist-cured concretes of Series 210, Fig. 9, p. 158 of the Report of the Director of Research of the Portland Cement Association (1928). The latter tests were begun in 1926. These data indicate that there was not much difference in strengths of concretes of Series A and B but the more recently made concrete of Series 210 was 30 to 50 per cent stronger than the concrete in the Wisconsin tests.

The calculated constitutions in per cent of the cements used in Series A and in Series 210 were:

Cement Used in Series	3 CaO. SiO_2	2 CaO. SiO_2	3 CaO. Al_2O_3	4 CaO. Al_2O_3 FE_2O_3	$CaSO_4$	MgO
A	28.8	40.2	13.8	8.3	3.6	2.5
210	33.0	39.0	12.0	9.0	3.0	2.4

These data indicate that the constitutions of these two cements were quite similar. However, the cement used in Series 210 had a residue on the 200-mesh sieve of 17.5 per cent and was finer ground than the cement used in Series A or Series B. Apparently the difference in fineness of the cements used was a more potent factor in producing the marked difference between the strengths of the earlier-made and the more-recently-made concretes than the difference in constitution. On the other hand, it does not seem possible that the marked discrepancy in the strengths of

TABLE 10—A COMPARISON OF THE AVERAGE COMPRESSIVE STRENGTHS OF CONCRETE IN THESE LONG TIME TESTS WITH PRESENT DAY CONCRETE

Series of Tests	$\frac{w}{c}$, by Vol.	0.94	0.94	0.94	1.36	1.36	1.36
	Age	28 da.	1 yr.	5 yr.	28 da.	1 yr.	5 yr.
Series A Tests* (1910)		1900	3400	4200	900	1900	2400
Series B Tests (1923)		2000	3000	4300			
Series 210 P. C. A. (1926)		3000	5000		1150	2400	

*In order to make the comparison on a basis of strength of 6 by 12-in. cylinders the values from series A, in which 6 by 18-in. cylinders were used, were increased 5 per cent and recorded in the adjoining columns.

these concretes was entirely due to the difference in fineness of the constituent cements.

CONCLUSIONS

1. Portland cement may be stored in a tight container, such as a milk can, for at least 20 years without suffering material change in its strength properties.

2. Tensile strengths of the 1:3 standard sand mortar briquettes cured in water over a 20-year period exhibited retrogression in strength after six months. In the five-year tests of Series B similar retrogression occurred after the 90-day tests.

3. The tensile strengths of 1:3 standard sand mortars cured outside and of those mortars cured in the cellar over the 20-year period increased approximately as the logarithm of the age, but the tensile strengths of the 1:3 standard sand mortars of the later tests cured outside decreased with age after the one-year period.

4. In the five-year tests, mortars of Janesville sand exhibited less retrogression in the tensile strengths of the 1:3 mixes than the similar standard sand mortars. The 1:1 and 1:2 mortars of Janesville sand showed little change in tenacity after curing outside one year.

5. Neat cement mortars exhibited great variations in tensile strength under the curing conditions imposed in both series of tests.

6. In many instances low tensile strengths were undoubtedly due to the presence of surface checks at the minimum cross-sections of the briquettes. In the water-cured specimens of the 20-year tests, leaching appeared to have weakened the tenacity of the 1:3 standard sand mortars.

7. The compressive strengths of neat cement mortars were much more variable than the compressive strengths of the sand mortars. Part of this variability in the neat cement tests of Series A was undoubtedly due to poor bedments and part was due to checking. The compressive strengths of the sand mortars in both series of tests continued to exhibit upward trends with age after the one-year period.

8. The absorptions of the individual specimens of a given concrete, in tests of Series B, were very uniform. There appeared to be little difference in absorption due to aggregate or brand of cement. Absorption appears to increase with the water-cement ratio.

9. All concretes of Series B exhibited losses in compressive strength due to 20 cycles of the freezing and thawing test. Loss in strength due to this treatment was greatest in the mixes having the high water-cement ratios and high absorptions and least in the rich mixes of low water-cement ratios and low absorptions. The aggregates and brands of cement used exercised little effect on resistance to freezing and thawing.

10. Curing the concrete out of doors caused in most cases small linear contractions which for 1:2:4 mixes were usually less than 0.01 per cent at the time measurements were made. The data indicate that either rich mixes or wet mixes contract more than lean or dry mixes. Specimens cured in the dry air of the laboratory contracted three to six times as much as similar specimens cured out of doors and after five years of indoor curing exhibited contractions of 0.04 to 0.05 per cent. Linear dimensional change appears to be directly proportional to the change in water content of concrete during curing.

11. The compressive strengths of the concretes cured in water or cured outside increased approximately with the logarithm of

the age. In the tests begun in 1910 this relationship has persisted through 20 years. In the tests of Series B it has held for 5 years of outside curing despite the evidence from the freezing and thawing tests. The effects of seasonal variations on the strengths of specimens tested during the first year is reflected in the strength-age curves of Series B.

12. The rate of increase in the strength of concrete with age is affected markedly by the humidity of the surrounding atmosphere. Concretes cured in the dry air of the laboratory for five years exhibited water losses of 3 or more per cent and little change in strength after three months. Concretes cured in the cellar where the humidity was less than outside but greater than in the laboratory exhibited a progressive increase in strength but a slower rate of increase than the concretes cured outside.

13. A comparison of the average compressive strengths of the concretes of a given water-cement ratio made in Series A (1910) and in Series B (1923) with later made concrete, Series 210 (1926), indicates that there was not much difference in the strength-producing properties of the cements used in Series A and in Series B; but the more recent tests demonstrate that the early hardening properties of present-day cements have been considerably increased. It is therefore questionable whether the compressive strength of present-day concrete will increase over a period of twenty years in accordance with the logarithmic relationship which obtained in the tests of Series A.

14. Although there was considerable difference in the calculated constitutions of the cements of Series B, the differences in the compressive strengths of the concretes made of these cements were small. Contrary to expectation, in these tests the concrete made of the cement, which was calculated to contain the highest proportion of tri-calcium silicate, was somewhat more absorptive, exhibited greater loss in crushing strength due to freezing and thawing, and slightly less crushing strength in the outdoor tests than the concretes of other cements containing lesser proportions of tri-calcium silicate.

15. Concrete made of crushed Lannon dolomite, though carrying a higher water-cement ratio, was slightly stronger than

that made of other aggregates. This superiority appears to have been due to surface characteristics.

16. With the consistency constant the data from tests of Series B show that the strength varies linearly with the cement-voids ratio.

17. The data from both series of tests confirm the general relationship between the compressive strength and the water-cement ratio. The tests of Series B indicate that this relationship is influenced to some extent by the character of the aggregate.

Readers are referred to the JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

Cinders as Concrete Aggregate

Report of Committee 203

BY EINAR CHRISTENSEN*, AUTHOR-CHAIRMAN

Because Mr. Christensen had intimate touch, as engineer and chemist, with the development of an industry dependent on cinders in the manufacture of cinder concrete masonry units, and had given a decade of study to cinder aggregates, their characteristics, properties and the special characteristics and properties which they give to concrete made from them—studies not confined to one factory or condition or locality but to many —he was asked to prepare, as Author-chairman of Committee 203, a report which should summarize available information on cinders as concrete aggregate. His report written, it was submitted to W. D. M. Allan, R. D. Bradbury, Miles N. Clair, Emil Praeger and M. J. Roach—serving as critic members of the committee. Their responses were not perfunctory; were for the most part, detailed and thorough-going—numerous revisions being suggested particularly by Messrs. Allan and Clair. Mr. Christensen acknowledges their assistance in a general strengthening of the report in its revision for publication. At Mr. Christensen's request, the report was also submitted to George A. Strehan. Mr. Strehan's response, too late for consideration prior to the present publication, takes issue in some details with opinions expressed in the report. It will appear later, together with further data and critical comment by critic members of the committee and others. In the report as published Mr. Christensen has had the last word. It does not completely reflect in every detail the views of all the critic-members of the committee, but in general has their approval as an excellent presentation of the subject assigned.—EDITOR.

*Consulting Engineer, New York City.

INTRODUCTION

A COMMON attitude towards cinder (or sand-cinder) concrete is expressed in the anonymous letter in the *News Letter* pages of this JOURNAL for November, 1929—the attitude, briefly, that here is a material to be viewed with suspicion. Its author appears in accord with a British architect (D. B. Butler, 1909) who wrote:

> Judging from the quality of breeze (here "cinders") which may daily be seen delivered to buildings in course of construction, presumably for incorporation in fireproof floors, etc., many a clerk of works must be imbued with a pleasing and childlike faith in the ability of portland cement to stick together any kind of rubbish.

While in the 20 years between these two opinions millions of tons of cinders have been used in making concrete, the skeptical attitude of the anonymous writer to this JOURNAL is somewhat justified. Few data have been published to throw light on the mystery of cinders and their use in concrete.

Text books and technical publications give very little information. Authors generally copy faithfully what has been written before—not altogether condemning cinder concrete, yet warning against its use.

The belief that cinder (sand-cinder) concrete is "dangerous" is not confined to the Anglo-Saxon world. *Die Tonindustrie Zeitung*, for April 25, 1908, reports that the German Department of Public Works recommends the prohibition of cinders as an aggregate for reinforced concrete. The concluding paragraph of the recommendation states:

> Die Verwendung der Schlacke zur Herstellung von sonstigem Beton, sogenannten Stampfbeton, wird nur da zuzulassen sein, wo die Gefahr, die tragende Eisenteile mit derartigem Beton in Berührung kommen oder Menschen von herabfallendem Putzmörtel getroffen werden können, ausgeschlossen ist.

In other words, cinder concrete must not be used where people may be hurt by the falling concrete. This is a sad specification. "Menschen von herabfallendem Putzmörtel getroffen werden können." It should be very unsafe to live in New York.

As cinder concrete is now being used in the United States and in Canada, in masonry units, equivalent to approximately 500

million brick per year, and sand-cinder concrete in floors and fireproofing of many noteworthy buildings, it is timely to attempt evaluation of the knowledge we have of these materials. It is discomforting that one large city permits what another flatly condemns—and that one engineer places thousands of cubic yards of a material which another believes doomed to failure.

The common attitude towards cinder concrete, particularly sand-cinder concrete made on the job, is not surprising. The contractor is not inspired by the looks of the aggregate, nor by a "1:2:5" specification. He pays less attention to this than to stone concrete; the aggregate is ordered over the telephone and passes directly from the truck to the mixer without much inspection. Twenty thousand cubic yards may be placed in one building in this manner. The result, sometimes good and sometimes bad, is generally not known in advance. Aside from extraneous matter the cinders look rather dirty and unreliable, because cinders are always wet, their color and the true nature of their particles being disguised.

To see what cinders look like, a sample should be dried and split by a standard sieve analysis. It will then be found, generally, that the aggregate contains a range of sizes and consists of particles which have sufficient strength to offer concrete-making possibilities. By the same simple process it is possible to observe the peculiar shape and cellularity of the particles. How many engineers make sieve analyses of cinders? Even while using larger quantities of them in the floors of one building.

Here the cinder unit industry has contributed to our knowledge. Factory-made cinder units could be studied readily with reference to their general characteristics—and that naturally suggested a study of the raw material and the quality factors. In preparing this report, the writer is relying on ten years' experience in the manufacture of cinder concrete building units. In the light of this experience he attempts to interpret the work of other investigators of cinders and the resulting concrete. It is hoped facts will be established tending towards the making of better cinder and sand-cinder concrete, and a better understanding of the material.

The committee has been directed to present in summary a survey of the whole field, from the nature of cinders to the characteristics of the resulting concrete. Within the last few years, certain problems of cinders and cinder concrete have been subjected to careful study; prior to that, the matter has been treated so broadly and repetitiously that a thousand references may well be condensed to ten or less.

While this committee's assignment is in reference to cinder aggregate, the report is concerned chiefly with the type of concrete in which cinders are used as the sole aggregate—the type of cinder concrete at present used most extensively in precast masonry units. This must be clearly understood, inasmuch as the term "cinder concrete" is frequently applied to "sand-cinder concrete" (1:2:5 or so) as used in floor arches and the like. The definitions used in this paper should be carefully noted. When the paper deals with cinder concrete it is mainly because the writer's experience has been with the cinder unit industry where cinders alone are used as aggregate. The cinders, however, are the important and typical constituent of both types of concrete—the nature of this raw material and the character which it gives to the concrete are of primary interest.

DEFINITIONS

The following definitions may not meet the wholehearted approval of all readers, but they will apply throughout this paper.

Cinder Concrete: A concrete made from an aggregate composed of cinders (commonly used in the manufacture of cinder concrete masonry units—the aggregate usually prepared by crushing).

Sand-Cinder Concrete: A concrete made from a mixed aggregate of sand and cinder (commonly used under the name "cinder concrete" in floors, fireproofing and the like—the sand generally considered the fine aggregate; the cinders sometimes screened but rarely crushed).

Cinders: The residue, containing more or less carbonaceous matter, from high temperature combustion of coal or coke— known as "industrial cinders," "boiler cinders" or "steam cinders," to the exclusion of the residue from domestic furnaces. Forced draft is generally applied in the combustion process.

Ashes: The residue, containing more or less carbonaceous matter, which remains after the incomplete combustion of coal, coke, wood, or the like. This is an inclusive term, most frequently applied to the residue from domestic furnaces.

Ash: The inorganic residue which remains after complete combustion of coal, coke, wood or other carbonaceous matter. This may be considered a "laboratory term."

Clinker: That part of the cinders which has been partly or completely fused during the combustion process. The clinker may be dense or more or less cellular.

The present report is primarily concerned with cinder concrete as here defined, and is concerned *only* with cinders as an aggregate. Cinders, as here defined, contain clinker (fused), ash (unfused), and unconsumed carbon (coke or coal). This raw material is obtained from industrial plants, gas works, locomotives, and similar sources. It is not to be confused with the residue from small domestic furnaces, which here comes within the category "ashes," and which is not used in making good concrete.

Confusion arises when the term "ashes" is also applied to the finer (partly unfused) particles of cinders. The physical character of these fines in the residue from high temperature combustion, is different from the soft and dusty ashes of the domestic furnace, and is generally of real value in making concrete. Perhaps some of the skepticism regarding cinder concrete resides in the fact that even learned men associate the material with the unpleasant refuse of their basement fire.

It is of primary importance to the maker of cinder concrete (or of sand-cinder concrete) to know that his raw material is coming from a source where a large amount of coal is burned. This usually entails uniformity of coal and uniformity and efficiency of firing, with the result that the cinders do not vary greatly from day to day. In most places, cinders have ceased to be a waste product; they are now considered a commodity, and a large producer takes care to prevent the use of his cinder pile as a general dump.

No general differentiation is here made between anthracite and bituminous cinders. While they differ in their physical character, they are both suitable for concrete when meeting the requirements here proposed for cinders. The "sulphur controversy" has given rise to the idea that anthracite cinders are superior, and certain specifications exclude bituminous cinders.

Evidence at hand does not justify this attitude. As a matter of fact, the writer prefers a well prepared bituminous cinder because the particles are usually stronger and because the major part of the combustible matter is usually present as coke.

In England, "breeze" and "coke breeze" complicate the terminology. The former is broader in scope than the term "cinders;" the latter applies to the small coke produced by coke ovens and gas works. The term "furnace clinkers" comes rather close to the definition of "cinders" proposed here. In Germany the word "Schlacke" applies to both cinders and blast furnace slag. When designating the former it should be "Kesselschlacke" —in case of the latter "Hochofenschlacke." The writer has encountered several misunderstandings (notably in patent cases) because of erroneous translation of these terms.

CINDERS

The inorganic residue from the combustion of coal is composed largely of silica, alumina, lime and iron, with smaller quantities of magnesium, titanium and alkali compounds. A small amount of sulphur compounds is generally present, mainly in the form of sulphates and sulphides. The combustion process is practically always incomplete, and the residue therefore contains carbon as coal or coke or both.

A complete chemical analysis of cinders is of very little value in judging the concrete-making possibilities of the raw material. The maker of cinder concrete is more interested in the physical character, and this may readily be judged by an experienced observer. An analysis, if at all desired, may be confined to a determination of coal, coke, sulphates, sulphides, and (perhaps) lime.

Cinders from well fired bituminous coal contain their carbon mainly in coke, formed by distillation on the grate, while anthracite cinders may contain considerable unchanged coal. Average percentages are of little interest, but the average combustible content of industrial cinders of the United States may well be close to 30 per cent. The frequently-encountered specification for cinder concrete, that the cinders "must be free from unburned coal," would eliminate most of the raw materials which have come

to the writer's attention. While such a specification is not justified, the carbon content of the cinders deserves, nevertheless, some attention.

Effect on Strength: When a specimen of cinder concrete is broken in compression, it will usually be observed that particles of coal are exposed as such, while particles of coke are fractured. This indicates that the smooth surface of the coal does not adhere to the cement, and that the coke has less strength than other particles of the aggregate. No definite relationship has been established between carbon content and strength—the main reason being that it is difficult, in comparative tests, to control the variable of consistency when varying the amounts of dense coal or porous coke.

Effect on Fire Resistance: It is natural to expect that the presence of coal and coke in cinders will reduce the fire resistance of concrete. Such an effect is, however, surprisingly small. Free combustion cannot take place when a piece of coal or coke is well protected in properly made cinder concrete. The only apparent effect is a distillation of bituminous coal when a certain temperature is reached—but that is a factor of minor importance in view of the high insulating quality of well-made cinder concrete. The effect is different when the concrete is so loosely packed, or the aggregate so void of fines, that fire may gain free access to the interior.

In factory-made cinder concrete units, the effect of carbon content upon fire resistance has been studied in a series of tests. The units were 8 x 8 x 16-in. hollow blocks, 2 cores, about 27 per cent air space, 2-in webs, having a compressive strength at 28 days ranging from an average of 825 lbs. per sq. in. of gross area for one batch to an average of 1156 lbs. for another. In preliminary tests, small panels (32½ in. square) were built of these units. The combustible content of the aggregate consisted, approximately, of ¼ anthracite coal, ¼ bituminous coal, and ½ coke—the extremes being an aggregate with about 20 per cent and another with about 75 per cent combustible content. The latter was prepared from nothing but coal and coke, the percentage being the result of chemical analysis, deducting ash. Considering these two extremes, the two panels had approxi-

mately the same endpoint (250 deg. F. above initial temperature of unexposed face) while the compressive strength after fire test had decreased 31.7 per cent with the lower carbon content, and 55 per cent with the higher carbon content. The panels behaved remarkably well, and even the "pure coal-and-coke block" did not "catch fire." The bituminous coal of this block suffered from distillation, evidenced by gas and smoke. At the same time, this distillation apparently retarded the heat flow through the wall. Later, test was made on a large panel (about 10 ft. x 11 ft.) which was subjected to a load of 80 lbs. per square inch during test. Three types of cinder blocks were included, made as before, and containing three percentages of combustible matter,—20, 35 and 45 (by weight of aggregate) respectively. After three hours fire exposure the endpoint was not reached. Next day the wall was loaded to failure, and failed in the 45 per cent carbon block at approximately triple load, or about 250 lbs. per square inch. On the basis of these data, the Underwriters Laboratories incorporated in their specifications for hollow concrete building units: "When cinders are used the average combustible content of the mixed fine and coarse aggregate shall not exceed 35 per cent by weight of the dried mixed aggregates."

Effect on Permanence: This has been given particular attention in England where instances have been reported of expansional movements of cinder concrete attributable to the presence of certain types of coal in the cinders. The writer has seen very few such instances in the United States, and only in one case (that of a poorly burned bituminous cinder) could it be ascribed to the presence of coal. Bearing in mind the foregoing definition of "cinders," it is believed that the phenomenon is of less interest here than in England where the material is less clearly defined.

The problem has been studied by D. B. Butler[5] and F. M. Lea.[15] The investigation by Lea is the most recent and the most comprehensive; it is part of the interesting work on cinders by the British Department of Scientific and Industrial Research.

According to Lea,[15] coals which are dangerous in concretes are so because of expansional movements which they produce during the setting period and early life of the concrete. The effect depends upon the nature of the coal; the amount present does

not by itself afford a test of the soundness of the material. The amount of moisture or of oxygen taken up by the coal are the characteristics to consider. With increasing rate of absorption the coal becomes dangerous.

Lea develops a test to determine the nature of the coal in cinders in its relation to the permanence of the concrete. It consists in making a 3-in. pat of 1 part binder (equal parts by weight of cement and plaster of paris) and 3 parts of finely ground (No. 76) cinders, gauged with water to a plastic condition. The pat is kept in a moist atmosphere for 3 to 4 hours, thereafter left totally immersed in water, and is examined after one to four days. If fine radial cracks appear, running from edge to center, or if the edges lift, or if the pat becomes loose, the cinders are unsound.

Cinders from high temperature combustion should contain very little coal of this nature, except due to accidents in firing the boiler. Such cinders, when bituminous, should contain the major part of their carbon in the form of coke. Anthracite coal appears to be harmless. Lea states: "If the coal has been treated to a sufficiently high temperature, its dangerous properties will be largely or entirely removed."

Sulphur Compounds

Various sulphur compounds may be present in cinders. In the unburned coal (if any) sulphur may remain as pyrites, marcasite or organic sulphur compounds; in the ash or clinker the sulphur may be present as sulphates or sulphides. Most cinders which have come to the writer's attention over a period of ten years contain less than 1 per cent of sulphur (S) in these compounds. They are present in the coarse as well as in the fine particles of the cinders.

When considering the possible effect of such combinations of sulphur, it should be borne in mind that they are not present in readily reactive form, but usually embedded in the coal or clinker or as an occasional large piece of pyrite. The behavior under such conditions is quite different from that of pure powders, and "laboratory deductions" do not always apply.

Corrosive Effect: It is well established that corrosion has occurred in several instances of reinforcing steel in cinder concrete

or of pipes and the like in cinder fill. Such instances, however, are many times outweighed in number by the performance of sand-cinder concrete or of cinder concrete building units where the behavior towards steel could be observed.

Corrosion of steel in cinder concrete has usually been explained as due to "sulphur," the supposition being that this element (seldom more clearly defined) by oxidation would form sulphuric acid which would then attack the steel. The chemical terms so appealed to many concrete engineers that this corrosion process became an axiom of the art. Instead of "sulphur" the word "acid" is frequently used to indicate that something is bound to happen when cinders are used—not merely corrosion, but staining as well.

It may be well, for a moment, to consider stone concrete. It is not uncommon to find corrosion of steel embedded in this material, but such instances are always explained by the fact that the concrete is poorly made or that the steel is improperly protected—a matter of mixing, making and placing—and no one would consider it necessary to assume chemically active elements in the aggregate. It would appear reasonable that the same explanation is at least to some extent applicable to concretes made with cinders as an aggregate.

Prof. C. L. Norton[1] came very close to this explanation in his well-known investigation of sand-cinder concrete. After pointing out that "sulphur" cannot be the cause in the "presence of the strongly alkaline cement," he concludes:

> There is one cure and one only: *Mix wet and mix well.* With this precaution I would trust cinder concrete quite as quickly as stone concrete in the matter of corrosion.

While Professor Norton absolves the "sulphur" of all blame and reaches the general conclusion that the protection of steel is a matter of the quality of concrete, he does attach some significance to the iron oxide, or rust, which may be found in cinders. He states that "a rust spot on reinforcement was invariably coincident with a void in the concrete or a badly rusted cinder." and concludes that this iron oxide may cause trouble. The explanation seems rather strange. It is far more plausible that the voids (permeability) were the sole cause, as they would allow moisture

to attack the reinforcement as well as the iron present in the cinders. The rust spots would naturally be coincident.

That the protection of steel in cinder concrete is primarily a matter of quality of concrete (mixing and placing) rather than a matter of some constituent of cinders, is supported by the evidence of several observers. Reference is here made to the investigation of sand-cinder concrete reported by Perrine and Strehan,[10] together with the discussion following that paper, notably by A. L. A. Himmelwright. Perrine and Strehan conclude as follows:

> Anthracite cinder concrete (1:2:5), well mixed, cast in a viscous to wet consistency, constantly stirred and mixed during placement, in such a manner as to coat the reinforcement thoroughly with mortar, will not cause the corrosion of embedded steel.

Millions of cubic yards of sand-cinder concrete have been placed in New York city. It is therefore significant when Rudolph P. Miller[7] (former Superintendent of Buildings, New York City) states (1911) that he has yet to hear of the first instance of corrosion of steel in this type of floors. While the cinders of New York are, most commonly, from anthracite coal, they nevertheless contain sulphur compounds to practically the same degree, and of the same nature, as cinders in other parts of the country. If corrosion is a matter of chemically active sulphur compounds, there would presumably be a large number of instances on record in New York. On the other hand, if it is a question of quality of workmanship, it is to be expected that the New York contractors would gain more than usual experience in the making of sand-cinder concrete, with the result that a better grade of concrete would be produced.

Factory-made cinder units represent well made cinder concrete —the cinders being crushed, well mixed with cement and water, and molded under pressure. These units are made throughout the United States and Canada—involving all kinds of cinders, anthracite, bituminous, Pennsylvania, West Virginia, etc., etc. The aggregate is all cinders, with no sand added. Therefore, if the corrosion of steel in cinder concrete were due to chemically active elements of the cinders, it would certainly have occurred somewhere and at some time in case of these cinder units. More so than in case of concrete where part of the aggregate is sand.

The writer has examined a great number of nails which for years had been embedded in this kind of cinder concrete—and also some cinder concrete lintels containing reinforcing rods. In no instance has he found sign of corrosion in the concrete. When a nail is driven into cinder concrete some of the aggregate particles are crushed and the dust is forced into adjoining pores, filling them and giving a firm hold on the nail. Fresh surfaces of cement are laid open, and if moisture gets into the concrete, further hydration takes place, with the result that the wound heals and the nail becomes cemented together with the surroundings.

Based on the above observations it appears unnecessary to assume any chemical reaction of sulphur compounds in order to explain occasional corrosion of steel in cinder concrete. It is possible that sulphates or other soluble salts of the cinders may accelerate rusting if moisture and air get into contact with the steel of the concrete. This is indicated by a recent investigation by F. L. Brady.[16] The method employed by Brady may, however, be subject to some criticism. The aggregates used included "coal clinker," "coke breeze," "furnace ashes" and "clinker from refuse destructor." These were ground to pass No. 20 mesh and made into mortar blocks (1:6). After one week, the mortar blocks were crushed to pass No. 10 mesh, and this dried mixture of aggregate and cement was used to make three series of tests. Steel strips were surrounded by the crushed mortars and various exposure to moisture and humid air were tried. As a basis for comparison of corroding power a comparable mixture was prepared using standard sand as aggregate. These tests showed (practically) an absence of corrosion in case of the sand mortar, and increasing corrosion in case of increasing amounts of sulphur compounds in the other mortars. This indicates that the corrosion process, under the conditions of the test, is accelerated by the soluble salts present as well as by the moisture adsorption, the latter being particularly large in cinder mortars.

In only one test series did Brady embed his steel in "cinder concrete"—and this concrete appears to have been of a somewhat unusual design. It is reported to consist of 1 cement, 2 "clinker," finer than No. 10 mesh, 4 "clinker" coarser than ¼-in. and finer than ¾-in., all parts by weight. It will be noted that inter-

mediate sizes are absent, indicating a rather inferior concrete. Furthermore, the steel was protected by only 1 in. of concrete. Corrosion occurred to varying degrees when the steel was examined after about two months, increasing with increasing content of sulphur compounds.

It should be observed that this effect of sulphur compounds is quite different from the effect commonly ascribed to "sulphur." It is a matter of accelerating the corrosion process when such process is possible because of improper protection of the steel in the cinder concrete (or in cinder fill). It is not a matter of an "acid attack" which would take place regardless of the quality of concrete. Such oxidation of sulphides to sulphuric acid is highly improbable, and even if acid were formed, certainly it would not attack the steel inasmuch as it would be neutralized immediately by the bases present in concrete.

The observations of the performance of reinforced cinder and sand-cinder concrete in the field are not upset by the above quoted laboratory test. The fact remains that millions of cubic yards of such concrete have given excellent protection to the embedded steel. The quality of the concrete (mixing and placing) is the determining factor. If the steel is not protected, corrosion is probably accelerated by the presence of soluble salts. Pipes and the like in cinder fill generally have no protection against air and moisture.

The steel must be protected. In floors, the sand-cinder concrete must have a waterproof top covering. The concrete should be well agitated and mixed while placing, and it is well to coat the steel with cement mortar. In masonry walls, cinder concrete building units must be protected against water forced in from the outside and the usual care must be taken in construction details at spandrel beams and the like where, otherwise, leakage may occur.

Expansional Effect: Dr. H. Nitzsche[13] has investigated the effect of sulphates in cinders on the permanence of cinder concrete. His work is concerned with the possible reaction between these sulphates and the lime of the cement. The reaction, if any, results in a volume change. Nitzche used a cinder containing "insoluble" $CaSo_4$ and soluble $MgSo_4$, K_2So_4 and Na_2SO_4. Start-

ing with a raw material rather high in these sulphates (containing .93 per cent "insoluble" and .81 per cent soluble) he reduced the percentages by repeated washing or increased them by adding the salts in powder form. The significance of tests involving the latter may be questioned, inasmuch as sulphates are not present in cinders in such readily reactive form. With an aggregate finer than ¼ in., and with 66.7 per cent retained on No. 16 mesh, he produced small Bauschinger prisms of 1 cement: 2 sand: 7 cinders. These prisms were measured for volume change under water, in air, as well as in sealed glass containers where the original moisture was maintained. On the basis of this investigation, Nitzsche reached the conclusion that for permanent concrete, the cinders must contain not more than 1.0 per cent $CaSO_4$ and 0.2 per cent soluble sulphates. He found the volume change to be considerably less when blast furnace cement (Hochofenzement) was used instead of portland cement.

Lea also investigates[15] the expansion of cinder concrete which may be ascribed to the sulphates. He goes further than Nitzsche in that he produces higher sulphate contents by fusing or sintering the admixed sulphates with the cinders. In this form, the effect of the sulphates is far less marked than as fine powders. Lea concludes that the limits proposed by Nitzsche are unnecessarily severe. He says of soluble sulphates (page 65) that "it is concluded that the average content of alkali, and magnesium, sulphates present in breezes and clinkers is small and may be neglected as of little importance." Of the effect of calcium sulphate he states (page 66) that "on the basis of these results it can be concluded that a limit of 0.4 per cent sulphate as sulphur trioxide (= 1.00 per cent SO_3 = 1.70 per cent $CaSo_4$) can be safely tolerated, and that in many cases much higher percentages are unlikely to cause trouble."

The ash and clinker may contain a small amount of sulphide and the unconsumed coal may contain Pyrite (FeS_2). The latter, however, decomposes at a temperature below 1290 deg. F., which is below the temperature of the furnaces considered here; it can therefore only be present due to accident in firing and the like. Lea produced oxidation and/or hydrolysis of calcium sulphide in cinder concrete when the sulphide had been added as

a fine powder—and he suggests a limit, in this case, of 0.4 per cent sulphide sulphur. However, when the sulphide was fused with a clinker it was found to be efficiently protected against hydrolysis by high pressure steam and apparently also protected against air oxidation. No effect was found with ferrous sulphide. Lee proposes a limit for "all sulphur other than sulphate sulphur" of .75 per cent S.

While the writer has observed occasional expansional movements of cinder concrete during its early life, he has never been able to identify definitely the cause of the phenomenon. The most common cause appears to be that of slow hydration of free lime (discussed later). Having no contradictory evidence on record, it is well to accept the limits proposed by Lea. Cinders, as previously defined will nearly always come well within these limits.

Lime

Small lumps of free lime may occur occasionally in cinders. Usually the lime is fused with the silica and alumina of the ash to form totally inert compounds. Small lumps of free lime may remain, partly hydrated or otherwise combined on the outside so that the result is a nucelus of CaO surrounded by a rather dense shell of hydroxide, carbonate, silicate or the like. Moisture may eventually penetrate this shell and hydrate the CaO. In a cinder concrete, the result is like that of "lime spots" in clay brick. The phenomenon is known as "popping." It is not a frequent occurrence and is evidently confined to certain types of coal. In cinder unit plants using steam curing, hydration lime spots will generally take place during the steaming process.

Contaminations

According to the definition, we are considering as a raw material only those cinders, which are the residue from high temperature combustion of coal and coke. If wood ash, pulp or the like are burned with the coal or coke on the grate, the cinders are contaminated. In selecting cinders for concrete, it is necessary to know something about how they are made. The specification should be written accordingly.

Cinders may be contaminated after they leave the boiler. Fortunately, most large producers of cinders have realized their

commercial value and the cinder pile is no longer considered a general dump. It is essential for the maker of cinder concrete to be sure that the cinder deposit is kept clean. The writer knows of a few scattered instances of caustic soda, molasses, and the like, being dumped with the cinders. These contaminations were readily discovered because the concrete would not harden. Particular attention is required where cinders are obtained from chemical plants.

Stray iron may be considered a contamination. It may occur as scalings from the fire place, as nails, bolts, grate bars, etc. In cinder unit plants this iron may be removed, with a fair degree of efficiency, by a magnetic separator. It does no general harm except when it gets into the equipment.

General Physical Character

The structural strength of the particles of cinders depends upon the nature of the coal and the temperature of burning. The clinker is the strongest part; it is formed by fusion of the ash constituents of the coal when subjected to the high temperature of the furnace. The fusing point of coal ash increases with the amount of alumina and the sum total of silica and alumina, and decreases with the sum total of iron oxide, lime and magnesia. A fusible ash will produce a dark clinker containing a relatively small amount of friable or fine material. A less fusible ash may yield laminated or honeycombed material which possesses less structural strength.

Because of the high fusibility temperature of anthracite, cinders from this coal do not contain clinker as thoroughly fused as cinders from bituminous coal similarly burned. The clinkers from anthracite are structurally weaker. The permissible maximum size of anthracite cinder aggregate is therefore usually less than for the average bituminous cinder.

Anthracite cinders usually contain more coal than bituminous cinders. While this coal is less objectionable than bituminous coal in concrete (it is chemically more stable and physically stronger) it nevertheless does not improve the concrete because of its smooth surfaces. As far as its effect upon the cellular structure of the concrete is concerned, the density of this anthracite coal is offset by the relatively high porosity of the clinker.

On the other hand, in bituminous cinders the carbon (present mainly as coke) increases the cellularity while the clinkers are relatively dense.

The weight depends upon the nature of the cinders (coke, clinker, etc.) and the grading from coarse to fine. Averages are of little interest. In one series of tests, a typical Eastern anthracite cinder (crushed to maximum ¾-in., F. M. 4.1) weighed 55 lbs. damp loose and 61 lbs. dry rodded per cu. ft. In another series, a bituminous railroad cinder weighed 53 lbs. damp loose and 60 lbs. per cu. ft. dry rodded. In a third series, a bituminous cinder from an electric power plant (crushed to maximum ¾-in., F. M. 4.5) weighed 64 lbs. damp loose and 71 lbs. per cu. ft. dry rodded. This cinder had about 20 per cent combustible content. With 60 per cent coke(weighing 42 lbs. damp loose) the cinder weighed 54 lbs. per cu. ft. dry rodded, showing the effect of the coke. With cinders from one source, the weight has been found to be of some value as a check on the uniformity of the product.

A SPECIFICATION FOR CINDERS

In proposing this specification for cinders, the writer has in mind both the raw material used in sand-cinder concrete (usually without crushing or other preparation) and that used in the manufacture of cinder building units (where it is further prepared by crushing, etc.) or in other types of cinder concrete.

Cinders, when used for concrete, shall be the product of high temperature combustion of coal and/or coke—known as "Industrial Cinders," "Boiler Cinders," or "Steam Cinders," to the exclusion of the residue from domestic furnaces. The cinders shall be well burned, free from foreign matter, and so graded from coarse to fine as to produce a cinder concrete (or sand-cinder concrete) meeting the strength requirement of the building code. The cinders shall contain not more than 35 per cent of combustible content by weight, nor more than 0.45 per cent sulphur (*S*) as sulphide, nor more than 1.00 per cent sulphur trioxide (SO_3) as sulphate.

The experience of the writer has shown that if the cinders meet the first part of the specification, they will also meet the requirements for unconsumed carbon (here adopted from the specifications of the Underwriters' Laboratories) and those for sulphur compounds (here adopted from the recommendations of Lea). It would then appear unnecessary to impose these further

tests on the producer. Accurate sampling of cinders for chemical analysis is extremely difficult—it is more conducive to the making of good concrete to expend time and effort in obtaining cinders from a reliable source, free from contaminations.

As fas as a general specification is concerned, a strength requirement is usually sufficient to govern the quality of cinder concrete, provided the raw material is "cinders" as here defined. It is not necessary to go into details of the physical makeup of the aggregate, or the design of mix, if the strength is produced. The specification as to maximum size naturally depends upon the purpose for which the concrete is used and should not be included in a general specification. The peculiar characteristics of cinder concrete, which depend upon its cellular structure, cannot be anticipated in a general specification for the raw material; they are developed to a varying degree dependent upon the admixture of other aggregate (sand) and the methods of making the concrete.

The writer knows of few suitable specifications developed by other investigators. Selecting a text book at random, the following specification is found:

> The standard specification for cinders states that they shall be of hard, clean, vitreous clinkers, free from sulphides, unburned coal, or ashes.

No such cinders are ever found. The author of this specification might as well have said: "Cinders are never to be used." The fact remains that cinders have been used with excellent results, although violating this and other specifications. The writer is of the opinion that a specification governing this or other materials, should be liberal enough to be practical. Otherwise it is not enforced.

JUDGING CINDERS FOR CONCRETE

Notwithstanding all exactness of the science of concrete making, judgment is still an important factor. When using cinders as an aggregate, it is well to be able to judge the quality of the raw material without having to resort to elaborate physical and chemical tests.

The manufacturer of cinder concrete building units usually contracts for his cinders, getting them from the same source

over a long period of time. He is more assured of uniformity than the user of an occasional few thousand yards of cinders for sand-cinder concrete in floors or fireproofing. The latter has to depend, largely, upon a cinder broker, and fluctuations in the cinder market make it difficult sometimes to be very exacting in the demands. The contractor may have to "take what he can get;" then the matter passes beyond the realm of exactness into the land of chance which cannot possibly be charted here. It is appreciated that sand-cinder concrete has occupied a peculiar position because of this element of uncertainty. No one would accept sand and gravel from "any pit," and if there is no choice among cinders, the writer is the first to advocate the use of something else.

It is certainly not good engineering practice to specify an aggregate by name only and then accept anything which may be pressed within the category. The foregoing specification is not as indefinite as that; the aggregate may, perhaps, not be judged to its second decimal—nevertheless it is fairly well defined.

An idea of the quality of a cinder may be based upon the following knowledge and observation:

1. Ascertain whether the producer has a steady and large output of cinders —employing forced draft or other means of efficient combustion. Much knowledge may be gained from the daily records of the producer, frequently furnishing chemical analysis.

2. Examine the pile of cinders to see if it contains any rubbish. If so, the pile may have been used as a general dump, and the cinders are not reliable.

3. From cinders from a chemical plant, obtain an average sample of about 1 cu. ft. Divide this by quartering, and boil a fraction with distilled water. Test the water extract with litmus paper. The paper should turn slightly blue. If it turns strongly blue—or red—the cinders should be analyzed to determine the cause.

4. Obtain an average sample as above. This sampling is done by taking shovel-fulls from edges, middle and top of pile. Mix well, quarter, re-quarter, and examine as follows:

4a. Dump these cinders in a bucket of water. The fine ash and the lighter coke will float on the surface, and the amount and the nature of these may be judged. A fair amount of each is desirable—the coke contributing to lightness and cellularity, the ash forming mortar with the cement.

4b. Drain the water from the bucket and compare the appearance of the washed cinders with that of the unwashed. The washing makes it possible to

determine with a fair degree of accuracy the relative amounts of coke, coal, fines, porous, flaky, or fully vitrified clinker. The grading may be observed.

4c. The structural soundness may be judged by breaking down the larger particles.

5. Determine the weight per cubic foot, damp loose, drained of free water. It should be not less than 35 lbs., preferably about 45 lbs. Pass it over a screen with ¼-in. mesh thoroughly shaking the screen. The amount retained on the screen should be at least 50 per cent by weight.

These suggestions may appear very crude to the engineer who is accustomed to deal with other aggregates where (theoretically at least) far more exact rules may be laid down. Surely, it is a far cry from judging cinders in a bucket full of water to obtaining a stone aggregate with a fully defined pedigree. Nevertheless, the use of the bucket has been very useful in revealing much regarding the nature of the cinders at hand. It is a matter of experience.

TESTING CINDERS FOR CONCRETE

In the foregoing the writer has suggested a shortcut for judging the quality of cinders for concrete. When it is a matter of examining the raw material for a cinder unit plant (which may well use 25,000 yds. or more per year), or for a large job of cinder or sand-cinder concrete (which may well require that amount in the floors alone), a definite effort should be made to obtain the cinders from one reliable source. They should then be subjected to more exhaustive tests.

If possible, cinders should be obtained from the source in question over a period of at least one week before signing contract. Daily test specimens should then be prepared in accordance with standard practice so that the uniformity and the quality of the resulting concrete may be determined. A week's average from a reliable source may well be taken as representative of the output.

It should not be necessary to emphasize that a trial run is of little value unless all factors are observed. The grading of the aggregate must be checked, the water content of the cinders must be known (it varies and is frequently very large), the amount of added mixing water must be determined, the method of mixing and making must be uniform. This is obvious for stone concrete; for some reason the rules are not always observed with cinder concrete.

The cinders should be submitted to a recognized laboratory for the following tests:

1. Determination of total combustible content. Nature of coal to be reported. Information may be gained from producer's records.

2. Examination of coal in accordance with method proposed by Lea and previously described. This test may be made upon the cinders or upon the coal picked by hand; the latter method will naturally emphasize results. The test is not necessary with anthracite.

3. Determination of total as well as water-soluble sulphates, quantitatively and qualitatively. Determination of sulphides.

4. Examination for impurities, visual as well as in water extract. Reaction of water extract to be reported.

5. Standard sieve analysis on representative samples to determine daily averages.

The results of this examination may then be judged in the light of previous discussion.

MAKING CINDER CONCRETE BUILDING UNITS

This chapter is concerned with concrete made with an aggregate of cinders only. It is primarily concerned with making this concrete in masonry units—at present the most important field for "self-contained" cinder aggregate.

The cinder concrete here considered is manufactured on pressure or tamping machines. It is of such consistency that the freshly molded unit retains its shape when carried from the machine, resting on a "pallet." The problem is therefore quite different from that of poured concrete, to be discussed later. The relatively dry consistency of machine-made cinder concrete units is made necessary by the manufacturing method. A type of concrete has been evolved which is not to be considered merely a compromise, but as having distinct advantages, among others a most interesting surface texture.

The standard unit is the 8 x 8 x 16-in. hollow block, containing two or three vertical cores, or from 30 to 40 per cent hollow space. This is the usual test specimen referred to. Most specifications require this unit to have a compressive strength, at 28 days, of 700 lbs. per sq. in. of gross area. It is well to bear this in mind when reading the following pages.

Before considering the various factors involved in making cinder concrete building units, we shall state the problem which

the industry is trying to solve. It is, briefly: To produce at the lowest possible cost a cinder concrete which combines cellular structure with the necessary structural strength. Strength, then, is not the sole criterion for quality—the cellular structure should be developed to the fullest possible degree.

Full understanding of the value of cinder concrete is not attained without realizing the peculiar character of the aggregate —its porosity. This porosity, and the corresponding limited strength, places certain limitations upon the use of the resulting concrete; but these limitations do not make the concrete inferior except when it is considered beyond its proper field of application. Within the field it has its distinct advantages.

While strength is not the sole criterion for quality, the required strength for the particular purpose must, of course, be met. As above stated, 700 lbs. per sq. in. is the usual requirement for hollow, load-bearing units. The manufacturer is primarily interested in obtaining this strength, and obtaining it economically. His first concern in his design of mix and in his method of making is that of obtaining the greatest possible yield per sack of cement for the strength necessary. Strength can be measured very simply—the other qualities residing in the cellular structure usually become secondary, but they should never be lost from sight.

Cinder Aggregate

The permissible maximum size is limited by the type of unit to be manufactured from the aggregate. The particles must be sufficiently strong to withstand the pressure or tamping—otherwise they are broken in the mold box of the machine, thereby creating new surfaces not coated with cement mortar. Generally speaking, the particle size should not exceed one-quarter to one-third the smallest dimension of the unit.

The cellular structure of cinder concrete is dependent upon the cells (sealed) and the pores (open) of the cinder aggregate. Of these, the sealed cells are particularly desirable, while the open, inter-connecting pores should be reduced to minimum size. The strength is dependent upon the strength of the particles and their bond. To get strength and proper porosity of the concrete,

grading of the aggregate is necessary. The unevenly shaped particles of the cinders contain numerous contact surfaces—these must be covered with mortar composed of cement and finer particles, and must be in contact. It is not desired, however, to fill the pores of the individual particles. Mortar lodged in these pores does not perform its function—on the contrary, it reduces cellularity.

The voids of a typical cinder aggregate, as used by the makers of masonry units, appear to approximate 55 per cent of the total volume of the loosely packed aggregate, when measured by the water displacement method. These "voids" represent not only the pores or pockets of the particles, but also open space between the particles. In making cinder concrete, such open space should be filled as completely as possible by cement mortar, to eliminate large channels through the mass. The pores or pockets of the particles are, to a large extent, filled with water which prevents mortar from getting in (unless the mixing process is *too* thorough). As far as the sealed and inaccessible cells are concerned, Nitzsche[11] found them to represent from 8 to 16 per cent of the total volume. These cells may be determined from the specific gravity of the finely ground material. A cinder aggregate should be graded from coarse to fine so the cement paste becomes most efficient. We are concerned with contact surfaces of rather irregularly shaped particles, and the problem is certainly less elementary than with stone concrete. The method of solving it is, of necessity, empirical.

The writer has conducted a large number of tests involving anthracite cinders; 8 x 8 x 16-in blocks, two-core, 30 per cent air space, 1 to 7 mix, were manufactured on a power press. All variables were controlled to give best results. The water content was varied according to porosity of aggregate, but was close to 10 gal. per sack of cement. For these particular conditions, a maximum strength cement ratio was obtained at a fineness modulus of 4.00 to 4.50 (¾-in.-0). Other tests by manufacturers of cinder units have generally supported this, but conditions vary and it is difficult to evolve universal rules. More recent tests, using a highly clinkered bituminous cinder, point towards a lower fineness modulus and a lower maximum size, but the data are not conclusive. The common maximum size of the

cinder unit industry is probably less than ¾-in. A large variety of units are made with thin webs which necessitate smaller maximum particles.

Nitzsche conducted similar experiments, but does not report his grading in great detail. He prepared prisms which he tested for transverse strength. The results are quoted as follows:

Aggregate No.	A	B	C
Sizes, by volume:			
.47″—28″ (12 mm—7 mm)	1 part	1.00 part	1.00 part
.28″—16″ (7 mm—4 mm)	1.25 part	2.00 part	1.00 part
.16″—08″ (4 mm—2 mm)	.18 part	.50 part	.50 part
.08″—00″ (2 mm—0 mm)	.33 part	.25 part	2.00 part
Weight per cu. ft.	60.6	56.8	68.8 lbs.
1:4.5 cinder concrete			
Weight per cu. ft.	85.0	75.0	81.8 lbs.
Porosity by absorp. test	28.5%	27.2%	31.0%
Transverse strength lbs. per sq. in.	2440	1720	1400
1:7.0 cinder concrete			
Weight per cu. ft.	78.1	70.0	74.9 lbs.
Porosity by absorp. test	32.6%	32.2%	30.7%
Transverse strength lbs. per sq. in.	1460	940	1170

Aggregate *A* is the best graded of the three and produces the strongest concrete. *B* contains too much material in the range from about ¼-in to 3/16-in. Its showing is particularly bad with the leaner mix (1:7) where the lack of fines becomes more pronounced in its effect. *C* contains too much fine material. In lean mixes these fines are of increasing importance. Note that this aggregate is rather heavy; evidently the weight is not a safe indicator of proper grading.

If a general rule were to be given for the grading of cinder aggregate to produce masonry units of strength 700 lbs. per sq. in. for maximum yield per sack of cement, a fineness modulus of 4.0 to 4.5 (¾-in.-0) might be suggested. It is important, however, to remember that so many other variables enter the manufacture, that it becomes necessary for the maker of cinder units to run his own tests to obtain data applying to his conditions. The variables not only involve those of the raw material, but also the machine used and general method of making and curing.

Experience shows that the run of the grate of cinders from one reliable source, crushed by a uniform method to produce the desirable maximum size, is generally surprisingly uniform in its grading. If this were not the case, it would be impossible for cinder unit plants to produce a uniform product. Separation of cinders into definite sizes, by screening, is very difficult because of the nature of this damp, sticky material. From a practical point of view, (not losing sight of cost), it is perhaps better to endeavor to keep the cinders together, i. e., to prevent segregation. This segregation is a real difficulty in many plants, and it is readily multiplied when screening is used. The manufacturer, then, relies on a uniform and reliable source of supply

Scientific grading by separation and recombination is desirable and deserves considerable attention. The above grading is proposed, mainly, as guidance in empirical selection of the raw material. Considerable control may be obtained by proper operation of the crusher.

Much has been said of the effect of the fine particles of cinders. The writer refers to his definition of cinders to exclude household ashes. Cinders from high temperature combustion do not contain any appreciable amount of fine, fluffy ash—they contain some of fine gritty material. These fines are rarely present in any undesirable excess—they are needed to make the otherwise harsh mix workable. A typical cinder aggregate usually contains less than 10 per cent finer than No. 100 mesh.

The average manufacturer of cinder units unfortunately does not perform a large number of sieve analyses. Realizing that the raw material, as run of the grate, usually contains the full range of particles, the writer has proposed as a simple rule that the aggregate should "split" 50-50 or 40-60 on ¼-in. sieve. In other words, the manufacturer should endeavor to obtain such division through selection of supply and operation of crusher. It is admitted that freak aggregates can be produced without violating this rule of division on ¼-in.—but the writer has rarely encountered them except in patent litigation.

Mixing Water

When considering the amount of mixing water to be used in making cinder concrete building units, it must be borne in mind

(1) that we are here dealing with a highly absorptive aggregate and (2) that this concrete mixture cannot be considered as being truly plastic.

A large amount of water is usually present in the cinders when they arrive at the plant, or in the aggregate when it reaches the mixer. The manufacturer describes his aggregate as "dry," "wet" or "damp", but few realize how much moisture is actually contained. Part of the water which may be absorbed by a cinder aggregate is readily drained off—but a considerable amount remains even after prolonged drainage. The following data may be of interest:

A bituminous cinder aggregate was obtained from a plant in which the cinders were washed before crushing; they were therefore very wet in the storage bin. The fineness modulus was 3.85.

	Moisture	Weight Per Cu. Ft. Loose	Rodded
	%	Lbs.	Lbs.
Aggregate as received	38.7	57.0	62.6
Dry cinders in wet aggregate		41.1	45.2
Dried aggregate	0.0	54.0	61.7
Same soaked 15 min. and then drained of free water	31.4	57.2	64.0
Dry cinders in this aggregate		43.5	48.7
Same drained 18 hours	9.2	47.7	56.2
Dry cinders in this aggregate		43.6	51.4
The cinders were then inundated:			
Dry cinders per cu. ft. of inundated aggregate		50.0	
Water per cu. ft. of inundated aggregate		33.5	

The cinders of the above test were drained by being placed in a canvas bag. After 18 hours they still contained 9.2 per cent of water. The aggregate, when taken from the storage bin, contained close to 16 lbs. of water per cu. ft. Used in a 1:7 mix, this would mean more than 13 gallons per sack of cement. In this particular plant, where the cinders were washed, no water was added in the mixer when the aggregate under consideration was used.

From the above data it is possible to determine the bulking of the cinder aggregate, loose:

54 lbs. dry cinder aggregate (F. M. 3.85) ..... 1.00 cu. ft.
54 lbs. dry plus 9.2% water ..... 1.24 cu. ft.
54 lbs. dry plus 31.4% water ..... 1.24 cu. ft.
54 lbs. dry plus 38.7% water ..... 1.31 cu. ft.
54 lbs. dry plus 67.0% water (inundated) ..... 1.08 cu. ft.

It is apparent that considerable bulking takes place. For the percentages recorded, this bulking is approximately constant within the limits of moisture usually found in cinders. When inundated, the aggregate did not return to the volume of the dry aggregate, presumably because of the presence of particles of low specific gravity. A "pushing apart" may take place.

The data emphasize the necessity of using care in applying results obtained with dry cinders in the laboratory to wet cinders on the job.

While the above data were obtained from a plant using washed cinders, the writer has found the moisture content, in cinder unit plants using cinders as they come from the source, to range from 10 to 25 per cent, ignoring extremes. The percentages quoted here are based on dry weight.

It is evident from all figures that a large part of the total mixing water, with this aggregate, is present in the cinders and that this must be taken into account. The writer is aware of many experiments in which only the water added in the mixer was measured. Inasmuch as the mixing water is an important factor, here as in other concretes, such experiments cannot be accepted.

The amount of mixing water to be used to obtain maximum strength-cement ratio, depends upon the nature of the aggregate. The following series involves 8 x 8 x 16-in. hollow blocks made on a tamping machine. The carbon content of the cinders was varied—starting with a well clinkered bituminous cinder and adding carbon in the form of ¼ anthracite, ¼ bituminous coal, and ½ coke. It was difficult under these conditions to judge the consistency of the mix, but it is believed that "best results" were obtained in each case.

Mix 1:7 Dry Rodded. Coal and Coke in Cinders—Mixed as Above Stated	U. S. Gallons Water/Sack of Cement	U. S. Gallons Water/Cu. Ft. of Aggregate
20%	13.0	1.85
35%	11.9	1.70
50%	10.8	1.55
75%	10.2	1.45
100% (aggregate consisting of coal and coke only)	9.1	1.31
Mix 1:9 dry rodded. 20% carbon in cinders	16.7	1.85

The addition of dense coal reduces the amount of mixing water to be used. The density of the coal here more than offsets the effect of the porous coke.

The amount of mixing water to be used in making cinder concrete building units must be determined in every instance under the conditions prevailing in a plant. For example, a test run in an Eastern plant, using anthracite cinders (F. M. 4.0) in making 8 x 8 x 16-in. hollow blocks (30 per cent air space) on a power press, gave the following results for a 1:7 mix:

Mixing Water	Compressive Strength 28 Days
7.8 gallons per sack of cement	624 lbs. per sq. in.
9.6 gallons per sack of cement	741 lbs. per sq. in.
10.5 gallons per sack of cement	835 lbs. per sq. in.

A later test run at this plant, using a somethat coarser aggregate (F. M. 4.5) brought best results when using 12 gal. of water per sack cement—additional water giving slump of the block and decrease in strength.

It may be of interest to consider another test run involving two types of mixes. Anthracite cinders of fineness modulus 4.0 were used throughout, same cement, 7 min. mixing time, same type of 8 x 8 x 16-in. hollow block (30 per cent air space), same method of curing. Three different types of block machines were used, but in each instance the one variable of water content was controlled with a fair degree of accuracy to produce maximum strength for the machine in question.

1. Power press:
   1: 7 mix: 10.4 gal. water per sack of cement.
   1:10 mix: 15.9 gal. water per sack of cement.
2. Heavy tamping machine:
   1: 7 mix: 11.8 gal. water per sack of cement.
   1:10 mix: 15.5 gal. water per sack of cement.
3. Light tamping machine:
   1: 7 mix: 11.0 gal. water per sack of cement.
   1:10 mix: 14.3 gal. water per sack of cement.

If a general rule were to be deduced from these tests on anthracite cinder aggregate of fineness modulus 4.0, it would be that a 1:7 mix requires about 10 to 12 gal. of water per sack of cement, and that a 1:10 mix requires about 15 to 16 gal. In the last

mentioned series we may figure the amount of water used per cubic foot of aggregate, loosely packed:

1. 1: 7 mix: 1.49 gal. per cu. ft. of aggregate.
   1:10 mix: 1.59 gal. per cu. ft. of aggregate.
2. 1: 7 mix: 1.71 gal. per cu. ft. of aggregate.
   1:10 mix: 1.55 gal. per cu. ft. of aggregate.
3. 1: 7 mix: 1.57 gal. per cu. ft. of aggregate.
   1:10 mix: 1.43 gal. per cu. ft. of aggregate.

Average: 1.56 gal. per cu. ft. of aggregate.

In the above described test on bituminous cinders of varying carbon content, it was found that the mixing water for the aggregate containing 20 per cent carbon (mainly coke) was the same in 1:7 and 1:10 mixes, viz., 1.85 gal. per cu. ft.

As previously pointed out, one-half or more of the total mixing water is generally present in the cinders as received from the original source. It is believed that it is desirable to have at least part of the water present in the aggregate when the cement is introduced—otherwise part of the binder may be lodged in the open pores of the particles.

In cinder unit plants, the amount of water to be added in the mixer is mainly a matter of judgment. Drainage of the aggregate in the storage bin causes considerable variation; a mix is used which cannot be judged by slump; it is thus for the mixer man to know. Frequent test runs are important. It is well to employ an automatic measuring tank for the water, even if it has to be adjusted several times a day; it does reduce a variable in the manufacture.

Method of Mixing

The paddle mixer with open stationary drum is the type most commonly used in cinder unit plants. The lightness of the aggregate and the harshness of the mixture favor this type of equipment.

Tests have shown that the length of mixing time has a marked influence upon the quality of cinder concrete. This is readily understood when considering the nature of the aggregate. The shape of the particles is such that the surface area is very large. The cement paste (mortar) must be distributed throughout. It is probable that the shape of the particles (and perhaps their lightness) also require thorough mixing in order to obtain the

desired distribution of all sizes throughout. Tests indicate that too prolonged mixing may lead past the desired maximum—probably because cement is forced into pores where it does not serve as binder.

The majority of tests on record indicate that an increase in strength of cinder concrete continues up to a mixing time of seven minutes. The following test is selected at random:

Aggregate: Anthracite cinders, fineness modulus 3.82.
Total mixing water: 86 lbs. per sack cement.
Proportions: 1 to 7 damp loose.
9 cu. ft. paddle mixer, speed of shaft, 20 R. P. M.
8 x 8 x 16-in. blocks, 30 per cent air space, made on power press and stored in air for 28 days to 1 year.
Mixing time: 2 minutes.
28 day strength: 676 lbs. per sq. in.
1 year strength: 838 lbs. per sq. in.
Mixing time: 7 minutes.
28 day strength: 835 lbs. per sq. in.
1 year strength: 950 lbs. per sq. in.

The speed of the mixer blades is naturally of importance. It usually varies from 18 to 25 R. P. M. in cinder unit plants. The writer has conducted tests with mixer speeds up to 48 R. P. M., but the results at this high speed were erratic due to difficulty in charging and discharging the mix as well as in keeping the materials in the mixer. For the type of mixer commonly used in these plants, a speed of 25 R. P. M. appears most satisfactory. This applies to several makes of mixers, apparently of about the same efficiency in mixing.

Method of Molding

It is not within the scope of this paper to discuss the large number of machines on the market. The dominating type in the cinder unit field today is the so-called "stripper" which ejects the unit either upward or downward after tamping or pressing in the mold box.

Naturally, the strength of the cinder concrete increases with increased packing, provided the particles of aggregate are of sufficient strength to avoid crushing in the mold box. The method of feeding is important. Generally, the unit (usually 8 in. high) is weaker in its lower part ("the pallet side") because of the

bridging effect of the rather harsh cinders. This is particularly true of units with narrow webs. Best results are obtained, apparently, when machines strip the unit downward, as this permits the web to be wider towards the bottom, reducing the bridging effect. Feeding the material in layers also helps (see report of Committee P-1 in the A. C. I. Proceedings 1928).

Method of Curing and Aging

The hydration of cement in cinder concrete proceeds as in other concretes. It is of importance here to consider the amount of water present in the freshly made concrete.

Because of the porosity of the cinder aggregate, a large water reserve is available for hydration of the cement, and because of the nature of the concrete, the water is slow in reaching the surface where it evaporates. We have previously quoted instances of machine-made units containing more than 10 gal. of water per sack of cement with a 1:7 mix and more than 15 gal. in case of a 1:10 mix.

This water reserve simplifies the curing method. The "green" units may well be stored at a temperature of 70 to 80 deg F. without danger of too rapid drying. As a matter of fact, it becomes a problem to get rid of the water if the units are to be ready for safe use at 28 days. The significance of "dry" units becomes apparent when considering the problem of volume changes.

In one series of tests, the writer found 8 x 8 x 16-in. cinder units to contain more than 2 lbs. of water per unit after 28 days storage in the laboratory. The drying is more rapid when the units are properly stored in the yard. It is of the greatest importance to have the yard covered with a shed in order to prevent wetting from rains. The general problem of piling, etc. is now being studied. Of course, too rapid drying is possible, due to exposure to sun or to certain atmospheric conditions. But this is a rare occurrence.

Manufacturing Methods Generally

While the mechanical operations of mixing, molding, curing and storing the cinder unit, closely resemble those of a typical concrete block plant, the manufacturer of cinder units is, in addition, confronted with the problem of turning raw cinders into

a suitable aggregate. We shall briefly consider this part of the manufacture.

A plant producing one million 8 x 8 x 16-in. hollow units will consume approximately 20,000 cu. yds. of raw cinders, depending upon the nature of the material. Inasmuch as the manufacture of the units is rarely synchronized with the supply of raw material (usually contracted for), it is necessary to provide reserve storage and facilities for handling. Ground storage and exposure to weather offer the advantage of getting the cinders washed by rain (weather permitting) more thoroughly than is possible by any mechanical washer. The content of soluble sulphates is reduced. Of course, drainage must be provided and the cinders must be drawn from the top of the pile; otherwise the aggregate may suffer from a concentration of soluble salts. While a clamshell bucket is very satisfactory for handling the cinders from the pile to the aggregate plant, it is generally too expensive to operate except in large plants; it requires a licensed operator. Drag line buckets have been used successfully, the main difficulty here being the wear on the cables sliding through the abrasive cinders. A system of portable belt conveyors is used in many plants. Others use a tractor with a scraper attachment.

The effort in cinder unit plants is to consume the cinders as soon as they arrive. They are usually delivered in gondola cars holding 40 to 80 cubic yards. In designing conveyors for cinders, raw or crushed, it must be borne in mind that the material is highly abrasive. Elevators should be slow moving of the continuous bucket type and the feeding so arranged that spilling is avoided as far as possible and that facilities are present for easy cleaning of the boot. Belt conveyors should be wide and well troughed and provided with skirt boards to prevent spilling.

The crushing method depends upon the nature of the cinders. Usually a primary crusher is employed for reduction to 1½ to 2 in., while a secondary crusher completes the reduction to the required aggregate size, say ¾ in. Jaw crushers and hammer mills are most frequently used as primary crushers. The former type serves well when there is not too much fine material present to cause clogging. When the latter type is used, the hammers should move at much less speed than for limestone. The grate

bars should be spaced about two inches. To attempt the complete reduction of raw cinders in a hammer mill (requiring high speed and close spacing) does not appear economical. Roll crushers are used for secondary reduction. A common type consists of two pairs of rolls, each of 16 in. diameter. If but one set of rolls is used, larger diameter is required to get the necessary nip. Of course, rolls may also be used in primary reduction if the rolls are properly spaced. The usual care must be used in obtaining the proper spring adjustment; perhaps this is particularly important here because of the difference in hardness of the particles of cinders. It is necessary to have the feed evenly distributed over the full width of the rolls—otherwise the wear becomes excessive in spots. Sometimes cinders are obtained of maximum size less than 2 in.; the reduction may then be obtained in one step, using a roll crusher.

The crusher units should always be operated in circuit with a screen. The crushers cannot be relied upon to control the maximum size of the aggregate, and the output should therefore be screened so that any oversize may be returned. It is not good practice to pass anything through a crusher that does not need crushing; the cinders should therefore first be screened. The two screening operations may be combined, at least in a medium size plant, by first elevating all cinders to a screen on top of the aggregate bin, letting the aggregate pass through and returning the oversize to the crusher, thereafter joining the raw cinders on their way to the screen. This type of closed circuit is most desirable; it saves the crushers, prevents undersirable breaking down of particles not in need of reduction, and insures against over-size getting into the storage bin. Of course, the layout of equipment of this type depends upon the nature of the raw material and the capacity of the plant.

Screening of cinders is, at the best, an incomplete operation. Cinders are always wet when they arrive at the plant, and fine particles adhere to the surface of, or are occluded in, the pores of coarser particles, and cannot be removed except after drying. The dampness of the cinders and the irregular shape of the particles cause blinding of even the best screen. It is not necessary, however, to obtain complete separation when the

object is merely to save the crusher from unnecessary work or to prevent too coarse particles from getting into the aggregate bin.

Revolving screens are very unsatisfactory for cinders. The screening efficiency is notoriously low, and blinding quickly takes place when damp cinders are introduced. Fast vibrating screens, mechanically or electrically agitated, produce the best results. The type of screen cloth to be used depends upon the nature of the raw material. Each case deserves expert attention.

One of the significant problems of a cinder unit plant is the prevention of segregation of the raw cinders and the cinder aggregate. This segregation is particularly pronounced when the aggregate is fed into a large bin from a bucket elevator. The thin stream discharged by the elevator segregates readily when striking the cone of aggregate built up in the bin. In several plants this is successfully overcome by having the elevator discharge into a dumping box of capacity ½ to 1 cu. yd. When the box is full, it dumps its entire contents into the bin, thereby avoiding the slow building up of the cone. It is further desirable to have the storage bins of small diameter (8 ft. or so), with steeply inclined bottom (about 45 degrees) leading to a central discharge opening. It is well to endeavor to keep the storage bin full.

Cinders, consisting of rough particles, light and damp, do not flow readily, and all chutes should be designed with a slope of at least 50 degrees. Otherwise segregation occurs.

MAKING SAND-CINDER CONCRETE

The writer has done little work with sand-cinder concrete, and must therefore rely, to a great extent, upon published data. Unfortunately, such data are incomplete. All that is generally reported are the arbitrary proportions and the compressive strength—with no information regarding the properties of the raw material, sand and cinders, or the design of mix (grading and water content), or the method of making. To know that a "1:2:5 sand-cinder concrete" developed a compressive strength of 800 lbs. per sq. in. is of no help to the maker of such material unless he knows more about its history. The wide variation in compressive strength reported certainly suggests the need of more data.

We are here concerned only with sand-cinder concrete as it is poured on the job, usually for floors and fireproofing. The problem is different from that of cinder concrete building units, where the concrete is machine made under pressure. Under such conditions, the harshness of the cinder concrete does not make it impossible to obtain uniformity. On the other hand, if the concrete is placed by hand, around reinforcing, it must flow more readily, and this is helped by the introduction of sand.

The main reason for the use of cinders in this type of concrete is the lightness of the aggregate. The main reason for the addition of sand is to produce workability. Strength and density are not the only criteria for quality in the field where sand-cinder concrete properly belongs. A certain strength is necessary, and sand is a helpful agent because conditions on the job cannot be controlled to the same extent as conditions in a plant. A certain density is necessary, but it is unfortunate that many investigators of sand-cinder concrete place density as a chief aim.

Cinder concrete may serve the purpose of sand-cinder concrete if the aggregate can be well enough controlled on the job and if the necessary care and effort are used in placing. The writer is familiar with several jobs of reinforced concrete (roofs, floors, beams) where a straight cinder aggregate was used—and large quantities of reinforced cinder concrete lintels are being produced by cinder unit plants, using consistencies ranging from "dry" to "wet."

The difficulty of rigid control of the cinders on the job suggests the use of sand as part aggregate. With a weight of 100-120 lbs. per cu. ft. (85-90 lbs. for cinder concrete) the sand-cinder concrete is still considerably lighter than stone concrete.

Of course, in addition to its lightness, sand-cinder concrete has other advantages inherent in the cellular cinders. These will be considered later.

Sand-Cinder Aggregate

The mixture of dense and cellular aggregates naturally possesses characteristics intermediate between the two extremes. What has been said about cinder aggregate will to some extent apply here.

Nitzsche[11] reports the following, in which he uses the cinder mixtures previously described under cinder aggregate, substituting for ¼ of the cinders, sand graded from about ¼ in. down.

Cinder *A:*

Sizes, by volume:

.47—.28 in. (12—7mm)	1 part	
.28—.16 in. (7—4mm)	1.25 part	
.16—.08 in. (4—2mm)	.18 part	
.08—.00 in. (2—0mm)	.33 part	
1 cement:4.5 cinders:	85.0 lbs. per cu. ft.	
	28.5 per cent pores (by absorption test)	
	2440 lbs. per sq. in., transverse strength.	
1 cement:3.4 cinders:		
:1.1 sand	95.5 lbs. per cu. ft.	
	26.3 per cent pores	
	2140 lbs. per sq. in., transverse strength.	
1 cement:7.0 cinders:	78.1 lbs. per cu. ft.	
	32.6 per cent pores	
	1460 lbs. per sq. in., transverse strength.	
1 cement:5.3 cinders:		
:1.7 sand	88.1 lbs. per cu. ft.	
	28.7 per cent pores	
	1310 lbs. per sq. in., transverse strength.	

The cinder aggregate appears to be well graded. Substituting sand for ¼ of these cinders does not improve this concrete; the weight increases (the sand being heavier)—the porosity decreases (the sand being denser)—but the compressive strength falls off. The reduction in transverse strength is more noticeable in the richer mix.

Cinder *B:*

Sizes, by volume:

.47—.28 in. (12—7mm)	1.00 part	
.28—.16 in. (7—4mm)	2.00 part	
.16—.08 in. (4—2mm)	.50 part	
.08—.00 in. (2—0mm)	.25 part	
1 cement:4.5 cinders:	75.0 lbs. per cu. ft.	
	27.2 per cent porosity (by absorption test)	
	1720 lbs. per sq. in., transverse strength.	
1 cement:3.4 cinders		
:1.1 sand	96.2 lbs. per cu. ft.	
	24.4 per cent porosity	
	2820 lbs. per sq. in., transverse strength.	

1 cement:7.0 cinders: 70.0 lbs. per cu. ft.
32.2 per cent porosity
940 lbs. per sq. in., transverse strength.

1 cement:5.3 cinders
:1.7 sand: 87.5 lbs. per cu. ft.
26.1 per cent porosity
1770 lbs. per sq. in., transverse strength.

This cinder aggregate is not well graded; it lacks finer particles. The addition of sand is therefore beneficial, and the strength increases notably in the leaner mix.

Cinder *C:*

Sizes, by volume:

.47—.28 in. (12—7mm) 1.00 part
.28—.16 in. (7—4mm) 1.00 part
.16—.08 in. (4—2mm) .50 part
.08—.00 in. (2—0mm) 2.00 part

1 cement:4.5 cinders: 81.8 lbs. per cu. ft.
31.0 per cent porosity
1400 lbs. per sq. in., transverse strength.

1 cement:3.4 cinders:
:1.1 sand: 91.2 lbs. per cu. ft.
27.3 per cent porosity
1600 lbs. per sq. in., transverse strength.

1 cement:7.0 cinders: 74.9 lbs. per cu. ft.
30.7 per cent porosity
1170 lbs. per sq. in., transverse strength.

1 cement:5.3 cinders:
:1.7 sand: 86.2 lbs. per cu. ft.
27.8 per cent porosity
1205 lbs. per sq. in., transverse strength.

This cinder aggregate contains too many fines. Substituting sand for ¼ of the aggregate (mostly fine) does not alter the strength much—it reduces the porosity and increases the weight.

It is apparent that in these tests the introduction of sand does not necessarily improve the strength of the concrete. Much may be accomplished with cinders alone, if the aggregate is well designed.

In their comprehensive paper on reinforced sand-cinder concrete, Perrine and Strehan[10] present some information regarding the aggregate used in tests. Cinders, and 8 x 16-in. cylinders of sand cinder concrete, were obtained from jobs in New York City.

The cinders (anthracite) were described as follows:

A: Cinders from apartment house zone (1914).
B: Cinders from zone between apartment house section and downtown.
C: Cinders from downtown section.
D: Cinders from Columbia University power plant.

The sand was the typical bank material obtained from the north shore of Long Island.

The grading of the cinders and sand was as follows:

	Percentage of Cinders Coarser than Sieve No.			
Sieve No.	A	B	C	D
1¼ in.	2.2	5.6	1.9	0.0
1 in.	2.8		3.7	5.7
¾ in.	6.6	19.7	11.7	9.7
½ in.	14.1	42.7	35.5	19.5
¼ in.	36.6	75.7	74.6	26.1
10	61.2	88.3	88.9	84.9
16				90.5
30	75.6	90.7	97.0	94.9
50	78.4	93.8	98.7	96.6
100	86.3	96.4	99.5	97.7
Estimated F. M. standard scale	4.29	6.02	6.00	5.14
F. M. of sand (approximate)	2.37	2.72	2.64	2.66

Combining cinders and sand for a 1:2:5 mix should produce the following fineness moduli of the combined aggregates:

	Fineness Modulus		
	Sand	Cinders	2:5 Mixture
A	2.37	4.29	3.75
B	2.72	6.02	5.08
C	2.64	6.00	5.04
D	2.66	5.14	4.43

In making (on the job) concrete specimens (8 x 16-in. cylinders) from these aggregates, the mixer used and the brand of cement were not the same, nor is there any record of the amount of mixing water. The results are not entirely comparable—yet it may be of some interest.

	A	B	C	D
F. M. of cinders	4.29	6.02	6.00	5.14
F. M. of 2:5 sand-cinders	3.75	5.08	5.04	4.43
1:2:5 sand-cinder concrete:				
Weight per cu. ft.	107	107	109	113 lbs.
Compressive strength lbs. per sq. in.:				
One month	407	818	980	787
Six months	933	1744	1478	
One Year	913	1465	1475	

Cinders *B* and *C* are particularly coarse, with about 75 per cent retained on ¼ in. The addition of sand should materially improve such aggregates. Cinder *A* contains a large percentage of fine material; it would have been interesting to know what kind of cinder concrete it would have produced; mixed with sand it did not make a very good concrete. Cinder *D* is rich in medium sizes (¼ in. to No. 10), and the addition of sand does not bring it quite on par with *B* and *C*

If it were possible to draw a definite conclusion from these tests (if other variables were really controlled), it would favor a fineness modulus of the combined aggregates of about 4.5 to 5.0. Taken together with the previous test by Nitzsche, it again appears that blindly adding sand to cinders does not necessarily mean good or better concrete.

The lack of fines in the above cinders, as compared to the aggregate discussed under cinder concrete building units, is partly to be explained by the fact that these cinders were not crushed (beyond a breaking of large lumps with shovels), and partly to be explained by the usual wetness of New York cinders which may lead to actual washing away of fines during transport. It is also probable that the samples of aggregate were not taken with a view towards eliminating segregation.

Perrine and Strehan report tests on *D* with the cinders finer than ¼ in. "screened out." These tests show that the screening resulted in a decrease in strength of 1:2:4, 1:2:5 and 1:2:6 sand-cinder concrete as compared to same with the cinders unscreened. It is not stated, however, whether the screening was done while the cinders were wet, nor how efficient it was. Knowing the difficulty of screening cinders, it is impossible for the writer to draw any conclusions without knowing more how the screening was performed.

Richard L. Humphrey,[3] and Humphrey and Losse,[4] made tests on sand-cinder concrete beams at St. Louis in which considerable care was used in making the concrete. A product of excellent quality and remarkable uniformity was obtained, and the data are illuminating.

In the first series of tests, cinders were obtained from a power house of the United Railway Co. of St. Louis; this was presumably

a bituminous cinder. Weight, 47.0 lbs. per cu. ft. The sieve analysis was as follows, maximum size 1¼ in.:

Retained on	¾ in.	½ in.	¼ in.	No. 10	No. 20	No. 30	No. 50	No. 100
Percentage	18.6	39.7	63.1	78.9	86.3	89.5	93.5	95.8

Fineness Modulus of cinders: 5.65. Fineness Modulus of Meramac River sand: 2.42.

The mixtures were proportioned by weight to obtain a 1:2:4 mix of dry loose aggregate. Due to an error in determining the moisture content of the cinders (it proved to be 23 per cent), the actual mixes turned out to be close to 1:2:5. A 1 cu. yd. mixer was used, and the materials were mixed 2 minutes dry and 3 minutes wet. Three consistencies were used, described as "Wet," "Medium" and "Damp." In each the total water was measured and expressed (including moisture in aggregate) in percentage of weight of dry mixed materials. Cement was here assumed to weigh 100 lbs. per cu. ft. In each case a number of 8 x 16-in. cylinders were prepared, the following data having reference to these cylinders:

Proportions By Volume	Proportions By Weight	Water % of Dry	Weight of Concrete at 13 Weeks	28 Days Strength Lbs. Per Sq. In.
"Wet Consistency"				
1:2:5.06	1:2.02:2.38	21.6		1,243
same	same	21.8	116.5 lbs.	1,000
same	same	21.6		1,000
			Average:	1,081
"Medium Consistency"				
1:2:5.06	1:2.02:2.38	19.4		1,114
1:2.06:5.40	1:2.07:2.54	20.4	115.6 lbs.	1,250
same	same	20.4		1,240
			Average:	1,201
"Damp consistency"				
1:2:5.19	1:2.01:2.44	19.0		1,055
same	same	same	112.5 lbs.	1,210
same	same	same		1,090
			Average:	1,118

The average compressive strength of these cylinders at 6 months may be reported as follows:

	Weight Per Cu. Ft.	Compressive Strength Lbs. Per Sq. In.
Wet consistency	113.9 lbs.	2,021
Medium consistency	114.3 lbs.	2,203
Damp consistency	113.7 lbs.	1,945

In the second series of tests, cinders were obtained from the same source. With a maximum size of 1¼ in., the grading was:

Retained on	¾ in.	½ in.	¼ in.	No. 10	No. 20	No. 30	No. 50	No. 100
Percentage	6.8	22.9	78.4	85.8	90.9	92.9	95.1	96.8

Fineness modulus: 5.69.

In this series of tests, sand-cinder concrete was prepared in the same manner as before, but the mixes were actually designed to be 1:2:4 by volume. This corresponded to about 1:2:2 by weight, the dry sand weighing about 100 lbs. and the dry cinders 50 lbs. per cu. ft. The total mixing water averaged 19 per cent of the dry materials for "medium consistency." Cylinders (8 x 16 in.) were prepared as before, and averaged as follows:

	Weight Per Cu. Ft.		Compressive Strength Lbs. Per Sq. In.
28 days	118.4 lbs.	High 1964 Low 1499	
		Average	1,647
13 weeks	119.4 lbs.		2,217
6 months	119.5 lbs.		2,525
1 year	119.7 lbs.		2,761

When observing the sand-cinder aggregate used in these tests it is interesting to find, first, how closely alike the cinders of the two series are. They were from one source, evidently producing a good grade of cinders. The fineness moduli are almost the same. Characteristic for both is a lack of fine material. Combining sand and cinders in the two general proportions produces the following:

	Fineness Modulus		
	Sand	Cinders	Mixture
1:2:5	2.42	5.65	4.72
1:2:4	2.42	5.69	4.60

The sand-cinder concrete of these tests was apparently of good quality. The strength for the 1:2:5 mixture is above those re-

ported by Perrine and Strehan, as representing "job made" New York concrete. It is unfortunately impossible to judge whether the sand cinder proportion was the most desirable for maximum strength/cement ratio, and it is also impossible to judge the density and other characteristics of the concrete. The many test specimens (representing the large number of reinforced beams prepared) show a most remarkable uniformity of strength. It is evidently quite possible to produce a uniform sand-cinder concrete if care is used in proportioning and if the cinders are from a reliable source. Other features of these tests will be considered later.

Some information regarding the effect of sand upon sand-cinder mixtures may be gained from tests made by C. M. Spofford and H. W. Hayward, as reported by Taylor and Thompson, "Concrete, Plain and Reinforced," 1916 edition, p. 328. We shall here omit the reference to the appearance of the cinders and the nature of the coal.

1:2:5 Sand-Cinder Concrete. 8 x 8 x 16-in. Prisms

% of Cinders Passing				Weight in Lbs. Per	28 Days Strength Lbs. Per
1 in.	¾ in.	½ in.	¼ in.	Cu. Ft.	Sq. In.
100	96	84	71	89	400
100	93	78	65	95	492
100	93	76	60	91	645
100	89	63	43	97	812
100	81	70	50	104	828
100	84	57	36	101	868
100	80	49	33	109	883
100	66	53	26	113	1088
100	83	47	26	111	1246
½-in. stone and coarse sand				145	1620

Different cinders were used in making these specimens and it is not known to what extent that affected the results. Nevertheless, it appears evident that there is a very definite relation between the grading and the strength, the latter increasing as the cinder-fines decrease from 71 to 26 per cent passing ¼ in. (the material through ½ in. decreasing in parallel manner from 84 to 71 per cent). It would be interesting to know how some of these mixtures would have behaved if no sand had been added. To add $^2/_5$

sand (by volume) to a cinder already having 71 per cent finer than ¼ in. will, of course, produce an altogether too fine aggregate.

Sand-cinder aggregate may be designed to produce definite results. It may not be possible to obtain the same degree of accuracy as with more expensive and better controlled aggregates, but an attempt should be well worth while.

Mixing and Placing

The purpose for which the sand-cinder concrete here considered is used, makes it necessary to obtain a mixture of greater plasticity than is required in cinder unit plants. It must be possible to work it around the wire mesh and rods of the floor slab, and distribute it well in the forms around the steel. On the other hand, it must not be so wet that the heavy sand and the light cinders segregate. There are few data available as to the effect of mixing water, mixing time or mixing methods on such concrete.

Perrine and Strehan state (page 580):

Cinder concrete should be mechanically mixed, for not less than 2 min., not leaner than 1:2:5, giving a minimum compressive strength, with standard cylinders at 28 days, of 800 lbs. per sq. in., and be in a wet to viscous consistency when cast.

Describing the making of floor slabs in the Columbia University laboratory, they state (page 550):

The process of mixing occupied about 2 min. per batch. The concrete was taken to the forms in wheel barrows, and when deposited was of a wet, viscous consistency. It was stirred and pushed into the spaces between the wires or rods with narrow pieces of scantling, the material being too wet to permit tamping, as the term is generally understood. An excellent, dense homogeneous concrete was thus obtained.

Note: A ¼-cu. yd. Blystone mixer was used.

As previously mentioned, Humphrey and Losse actually report the amount of mixing water used. Accepting the statement that "the water is figured in percentage of total weight of dry materials" to mean that the percentages are based on the weight of cement plus aggregates, the amount of water may be readily figured. Humphrey describes what is meant by "wet," "medium" and "damp" consistency, but it is not clear whether his descrip-

tion applies equally well to granite, gravel, limestone and sand-cinder concrete of his investigation.

It is apparent, immediately, that there is not much difference between the amounts of water used in producing the three consistencies. Apparently, the sand-cinder concrete mixture is just as sensitive to a pound of water at the point of proper consistency as other concrete mixtures. This is in accordance with the writer's observations of wet (almost plastic) mixtures of cinder concrete.

In the first series (1908) the mixing water (including absorbed moisture in the aggregates) is reported to be (in U. S. gal.).

1:2:5.06 sand-cinder concrete
"Wet" consistency:
14.1 gal. per sack cement.
1:2:5.06 sand-cinder concrete
"Medium" consistency:
12.6 gal. per sack cement.
1:2.06:5.40 sand-cinder concrete
"Medium" consistency:
13.7 gal. per sack cement.
1:2:5.19 sand cinders concrete
"Damp" consistency:
12.4 gal. per sack cement.

In the second (1911) series, the mixing water was about:
1:2:4 sand-cinder concrete:
11.4 gal. per sack cement

On the average, the amount of water for the 1:2:5 mix is about the same as the water used in a 1:7 cinder concrete mix as used in the manufacture of cinder concrete building units. The sand-cinder concrete, with the dense sand as part of the aggregate, has, of course, a consistency quite different from that obtained with straight cinder aggregate.

In the tests by Humphrey and Losse it is apparent that, for the particular cinder used, the amount of mixing water (including the large amount of moisture in the aggregates) should be about 13 to 14 gallons per sack of cement for a 1:2:5 mix, and about 11 to 12 gallons per sack of cement for a 1:2:4 mix. The mixing time used here (5 minutes total) should be sufficient.

Most authors agree that the sand-cinder concrete mixture must be wet. It can unquestionably be too wet. The test by

Humphrey indicates that (see above). The temptation of adding altogether too much water to stone concrete is well known—the same temptation is present here, perhaps to a greater extent. The cinders make the mixture harsh, yet the mixture can be properly placed without approaching the "liquid" form. The writer has observed that for reinforced cinder concrete, with no sand, carefully placed in cinder unit plants where the workmen knew a bit about the material they were handling, the sand-cinder concrete can be made so wet that it flows without any effort on part of the workman, but there is then great danger of segregation of the two aggregates, sand and cinders, because of the different specific gravities.

Stone concrete is gradually getting on a basis of rational design. The writer can see no reason why sand-cinder concrete should not follow. To specify arbitrarily a mixture of 1:2:5 just because one of the aggregates is not known to the second decimal is rather absurd. The engineer should know a little about his aggregate beyond the fact that it is "cinders," and he should run regular trial batches to determine just what he can accomplish with the material at hand.

On the Making of Sand-Cinder Concrete Generally

It is more difficult here to watch the cinders than in a cinder unit plant which usually contracts for a year-round supply. The storing of cinders on the job is usually limited to small quantities; generally the material is conveyed directly from the truck. The problem, however, is the same as in the cinder unit plant, viz., to prevent segregation.

The difficulty (if not impossibility) of crushing and screening on the job makes it necessary to obtain a cinder that does not contain much in excess of 1 in., or which is readily broken by shovel or in the mixer. The anthracite cinder of New York is of this type, as indicated by tests quoted.

Most of the data here quoted deal with 1:2:5 sand-cinder concrete, the apparently typical New York mixture. While it cannot be judged without knowing the nature and grading of the cinders to which the sand is added, it must be considered as being, generally, too lean. Attention should be directed to richer

mixtures, and to the proper gradation of the combined aggregates. A too harsh sand-cinder concrete leads to the use of too much water to get it to flow into place—with the result that the cement paste is diluted and segregation is likely to occur. The deficiencies of 1:2:5 mixtures should not be considered a reflection upon better designed mixtures of sand-cinder concrete.

The writer is convinced that there should be a considerable market for a prepared cinder, crushed and graded to meet definite requirements. This is probably the only way in which rationally designed cinder or sand-cinder concrete mixtures may be obtained. The chief obstacle is perhaps the fact that the contractor, faced with a "1:2:5" specification does not care what the "5" represents. Until this is remedied, sand-cinder concrete will not emerge from its present twilight zone.

Note on Job-Made Cinder Concrete

The foregoing pages may give the impression that job-made concrete (structural concrete), utilizing cinder aggregate, must of necessity contain sand. The writer has discussed sand-cinder concrete for this purpose, mainly because this type of concrete has been so generally used. He has pointed out, however, that the admixture of sand does not always improve the quality of the concrete—it all depends upon the nature (the design) of the cinder aggregate.

Since this report was first written, the writer has become familiar with some of the data obtained by the Thompson & Lichtner Co., Boston, in their work on structural, straight-cinder concrete. While these data (furnished by Miles N. Clair) are not incorporated in the present paper, it is hoped that they will be available in discussion of this report.

Cinder concrete mixtures of well defined slump were here produced. Small difference was found in the workability of cinders and other aggregates if they were all of the same grading. This applies particularly to mixtures 1:2:4 or richer. The data suggest as the most desirable a straight line grading on the semi-log plot with an allowable percentage of fines somewhat greater than is usual for sand. Ten per cent are permitted finer than No. 100 mesh. For usual maximum size (¾ in.) about 50 per cent should be retained on No. 4 mesh.

CHARACTERISTICS OF CINDER AND SAND-CINDER CONCRETE

In the preceding discussion of cinders it has been pointed out that the physical character of this aggregate is quite different from that of such aggregates as sand or stone, and that the resulting concrete should not be measured by the usual standards of strength and density. The field in which cinder, or sand-cinder concrete is used should be distinguished from the general field of stone concrete. The cinders should not be considered a substitute material—they produce a concrete possessing some highly desirable characteristics. These characteristics will briefly be reviewed here in the general discussion of the material.

Strength

First, concerning cinder concrete.

The effect of raw material and method of making upon the resulting concrete have already been discussed. The compressive strength will, of course, always depend upon the size and shape of test specimen, moisture content, etc. Hollow cinder concrete building units are usually made for a compressive strength of not less than 700 lbs. per sq. in. of gross area at 28 days. Because of many variables in the present day manufacture, it is necessary to aim at about 900 lbs. in order to be well above the minimum.

The compressive strength decreases with increasing moisture content. One series of tests on 8 x 8 x 16-in. hollow block of average compressive strength 827 lbs. per sq. in. showed a decrease in strength of 12 per cent after 48 hours immersion in water (tested wet). Another series of tests on brick of average compressive strength 4920 lbs. per sq. in. (tested flatwise) showed a decrease of 36.7 per cent after 48 hours immersion. The brick were denser than the block—evidently the effect of the water is more pronounced with lesser porosity.

Accepting cinders as an inert aggregate, it is natural to expect an increase in strength with age. Such increase is evidenced by many tests. When subjected to freezing and thawing treatments (described later), cinder concrete generally shows an increase in strength after completion of the test, obviously due to increased hydration of the cement during the submersion period.

There are few tests on record to show the strength of walls constructed of cinder concrete masonry units. They indicate a high wall efficiency (54.7 to 75.8 per cent), which is to be expected when considering the surface texture of machine-made cinder concrete. These same tests (on hollow cinder block piers, 54 in. high by 24 in. wide by 8 in. deep, laid in 1:3 cement mortar) showed a modulus of elasticity of the pier of 698,000 lbs. per sq. in. based on the intensity of stress on the gross cross section, and 1,035,000 lbs. per sq. in. based on the net section of 130 sq. in.

The transverse strength of cinder concrete building units is of interest chiefly in brick shapes, the larger unit rarely being exposed to tensile stresses in the wall. The transverse strength of cinder concrete brick increases with the compressive strength as indicated by the following representative tests: Cinder brick—standard size.

Compressive Strength	Modulus of Rupture
1221 lbs. per sq. in.	379 lbs. per sq. in.
1890 lbs. per sq. in.	496 lbs. per sq. in.
3035 lbs. per sq. in.	581 lbs. per sq. in.
4920 lbs. per sq. in.	765 lbs. per sq. in.

Second, concerning sand-cinder concrete.

It is not within the scope of this report to consider the design and behavior of reinforced sand-cinder concrete. The reader is referred to the papers by Humphrey, Guy B. Waite and Perrine and Strehan.

As to the concrete itself, Perrine and Strehan observe that:

> The crushing strength of standard (8 x 16″) cylinders of good cinder concrete (sand-cinder) as used in New York is between 800 and 1,000 lbs. per sq. in. at 28 days.
>
> The modulus of elasticity, E, under working loads, determined from standard cylinders, is about 1,200,000 lbs. per sq. in. for average concrete at 28 days.

As previously pointed out, Humphrey produced a considerably better sand-cinder concrete in his tests than the New York job concrete cited above. His 1:2:5 concrete ranged well above 1,000 lbs. at 28 days (8 x 16-in. cylinders), and his 1:2:4 concrete averaged 1647 lbs. at 28 days and reached an average of 2761 lbs. at 1 year. In the latter series of tests he produced 1:2:4 sand-gravel concrete of 3175 lbs. per sq. in., 1:2:4 sand-limestone con-

crete of 2492 lbs. per sq. in., and 1:2:4 sand-granite concrete of 3054 lbs. per sq. in.,at 28 days.

Humphrey and Losse report the initial modulus of elasticity (second series) to be:

1:2:4 sand-cinder concrete: 21 8 x 16-in. cylinders: 28 days:

High:	2,320,000 lbs. per sq. in.
Low:	1,180,000 lbs. per sq. In.
Average:	1,610,000 lbs. per sq. in.

For the 1:2:5 mixture, medium consistency, he reports the following initial modulus of elasticity:

At 28 days	Average: 1,352,000 lbs. per sq. in.
At 6 months	Average: 1,618,000 lbs. per sq. in.

Weight

The weight of cinder and sand-cinder concrete depends on the nature of the aggregate, design of mix, method of making, moisture content of the concrete.

A recent survey of the cinder concrete building units made by 28 plants gave the following results, 3 units from each plant being tested:

Weight of dry cinder concrete per cu. ft.

High value:	98 lbs.
Low value:	74 lbs.
Average:	87 lbs.

In one plant, the cinder concrete was found to weigh 106 lbs. per cu. ft., but this was due to an exceptional cinder, very high in iron. The above variation is wide—but these plants use widely different methods of making, in addition to representing cinders from all over the United States.

Perrine and Strehan report weights per cu. ft. of 1:2:5 sand-cinder concrete (using New York anthracite) ranging from 107 to 115 lbs. Humphrey reports his 1:2:5 sand-cinder concrete (using, presumably, a bituminous cinder) to weigh from about 112 lbs. to about 117 lbs. per cu. ft., while the average for his 1:2:4 sand-cinder concrete is 118.4 lbs.

Absorption and Allied Phenomena

It is still common belief that the old fashioned absorption test, in which the water absorption is determined by submerging the

specimen in water for a certain length of time, and is calculated in percentage of the dry weight, is indicative of such qualities as resistance to moisture penetration, frost action, etc. It would appear obvious, however, that this absorption test is of value only when it is representative of the actual conditions to which the material in question may be exposed. It may be of value with materials to be used under water—but its value is not clear for materials used in ordinary wall construction.

In the present paper, cinder concrete is considered as an intentionally porous concrete—in other words, a concrete utilizing a cellular aggregate in order to obtain certain characteristics—and not a concrete merely using cinders because they are cheap or because they are light. The value of the usual absorption test is questioned in a field where this porous concrete has proven its place. Cinder concrete does not belong where great density is needed. This may appear obvious, although it is not yet generally realized.

The immersion test gives no information regarding the behavior of cinder concrete in the wall of a building. Capillary attraction is the force which usually moves moisture through a wall, but that force is not measured by the standard test. Frost action depends upon the distribution of moisture throughout the wall, and upon the nature and distribution of the pores of the material, and perhaps upon other factors, but the immersion test gives no information about these matters.

It may be mentioned in passing that it is still quite common to specify an absorption limit for any kind of concrete in percentage of dry weight. How meaningless this is when including lightweight concrete is obvious. Assuming the limit to be 10 per cent by weight, 1 cu. ft. of stone concrete (140 lbs.) is permitted to absorb 14-lbs. of water, while 1-cu. ft. of cinder concrete (85 lbs.) must not absorb more than 8½ lbs. This has been remedied in most modern building codes; if a limit is considered necessary, it should probably be stated in lbs. of water absorbed per cu. ft. of concrete.

Frost action is usually determined in the laboratory by saturation of the material in question followed by alternated freezings and thawings. Such a test on cinder concrete building units is

reported and discussed in detail in the 1928 *Proceedings* of the American Concrete Institute. The freezing test specimens were soaked in water, then placed in the freezing chamber until frozen solid. After freezing for a period of 5 hours or more, during which time the temperature in the freezer was reduced to an average low value of 21 deg. F., the blocks were moved to a tank and thawed in water at 140 deg. F. for one hour. The blocks were cooled in water and put back into the freezer in the saturated condition, and the freezing and thawing was continued in like manner until 100 reversals had been made. The results are summarized as follows:

8 x 8 x 16-in. hollow cinder block.
Average compressive strength at 28 days (3 units): 1030 lbs. per sq. in.
Average dry weight at 28 days (3 units): 32.08 lbs.
Average absorption, percentage by weight at 28 days (3 units) 20.35 per cent.
Blocks normally cured, tested at 109 days:
Average compressive strength (3 units): 1042 lbs. per sq. in.
Average weight (3 units): 31.80 lbs.
Average absorption, percentage by weight (3 units) 20.12 per cent.
Blocks submitted to 100 freezing reversals, tested at 109 days:
Average compressive strength (6 units): 1123 lbs. per sq. in.
Average dry weight before freezing (6 units): 32.18 lbs.
Average dry weight after freezing (6 units): 31.75 lbs.
Average absorption, before freezing (6 units): 19.83 per cent.
Average absorption, after freezing (6 units): 23.12 per cent.

It will be seen that the blocks subjected to 100 freezing reversals showed an average compressive strength which was 7.8 per cent greater than the strength of the blocks normally cured. The absorption increased 16.6 per cent of the value before the freezing reversals, and the weight decreased 1.34 per cent. The increase in strength may be considered due to the warm water thawing treatment; the increase in absorption must be due to the opening, by the force of the freezing water, of cells which were previously inaccessible to water, as well as to other changes in pore structure. The significant facts are that the loss in weight was only 1.34 per cent and that the blocks showed no visible sign of cracking or spalling. No difference could be noted between the fractures from the compressive strength test of the frozen blocks and the blocks not frozen.

The writer has supervised several freezing and thawing tests in the last ten years. Qualitatively, the results were usually the same. An exception, however, is of interest. It involved cinder brick, and may be compared with another cinder brick made by the same plant, but from a different mix. The tests were carried out at Rochester, and at New York respectively. The results are summarized as follows:

	"Rochester" Brick	"New York" Brick
Freezing reversals	42	20
Compressive strength of brick not frozen	3290 lbs. per sq. in.	4920 lbs. per sq. in.
Absorption, by weight	15.8%	6.34%
Absorption corrected for weight according to A. C. I.	11.5%	4.65%
Change in strength due to freezing reversals	2.2% increase	24.1% decrease

Based on the recognized tests for strength and density, the "New York" brick is far superior to the "Rochester" brick, but the recognized freezing and thawing test showed the Rochester brick to be far superior. This may be compared to another test, made in 1922. The writer prepared cinder brick (using the same machine as above) from cinders crushed and screened to pass ⅛-in. mesh. At 28 days the brick tested 3,035 lbs. per sq. in. average, and had an absorption of 10 per cent—thus passing most quality requirements. When subjected to freezing and thawing test, however, they completely disintegrated within 20 reversals.

It would appear from this that with cinder concrete the absorption test does not give any information regarding the resistance to frost action, unless, perhaps, a minimum requirement should be considered. 15-20 per cent absorption by weight appears to be desirable. The action in freezing tests may be explained in two ways. It may be assumed that a highly cellular cinder concrete cannot be completely saturated with water, but that imprisoned air in the open pores (or the closed cells when forced open) permits expansion of freezing water without danger of fracture or disintegration. It may also be assumed that porous cinder concrete, in a standard freezing test, permits the water to drain off so rapidly that the concrete is far from being saturated when placed in the freezing tank. This may be taken to imply

that the standard test should be changed to insure complete saturation at all times—but would that be more truly representative of actual conditions? A material that drains rapidly in the wall, should also be permitted to drain rapidly in a test designed to determine its "wall behavior."

We shall next consider moisture movement through the wall. The usual absorption test is supposed to tell whether or not a material will produce a dry wall. Surely, that supposition is not justified unless we know something about the manner in which moisture travels through the wall. It is safe to assume that 100 per cent density will keep moisture out and that 100 per cent porosity will let moisture pass, but the fixing of a limit between these two extremes is not a matter of round-table discussion.

Suction is an important factor. A driving rainstorm does not ordinarily penetrate an 8-in. wall by its own force; the water is usually drawn by capillary force. It has been claimed for cinder concrete that this capillary force is very low because of the widening out of open channels into air pockets created by the porous cinder aggregate. The dependancy upon the porosity of the cinder concrete has not yet been proven, although most cinder concrete building units appear to have a rather low capillary attraction for water. Tests on cinder concrete and sand-cinder concrete indicate that the shape and distribution of the pores are the important factors, and that these cannot be judged by simple tests for porosity or absorption.

The moisture movement in a wall also involves the rate of drying. It is important that the wall should dry readily after getting wet, otherwise a magazine of water may be built up in the wall, leading to saturation. Of course, the drainage in a porous material helps in removing the water. There is apparently no definite connection between the rate of suction and the rate of drying; the distribution of the fine pores may be of chief importance. The usual absorption test gives no guidance.

The absorption test, or a simple determination of porosity, apparently gives no information regarding the true significance of the cellular structure of cinder concrete. We have previously shown (when discussing cinder aggregate) that there is no relation between porosity (determined by water absorption) and com-

pressive strength. This is also apparent from a number of tests conducted by the writer. The cellular structure, however, is a most important feature of cinder concrete—its chief characteristic—and should be first on the research program of the industry.

The introduction of a denser aggregate in sand-cinder concrete naturally affects the cellular structure. Writers on the subject have frequently indicated that it may be destroyed completely, and have placed the density of stone concrete as the goal for sand-cinder concrete. It is this, more than anything else, which has given cinders the appearance of being a substitute aggregate striving for something it cannot reach. A realization of the true nature of cinders should bring about an understanding of just what this aggregate will accomplish. As previously pointed out, the chief reason for the introduction of sand with cinders in the making of poured concrete is that of workability under prevailing conditions; it is not that of imitating stone concrete in other respects.

Volume Changes

The writer has previously mentioned the expansional movements which may be caused by certain types of coal or by sulphur compounds present in the cinders. It was pointed out that this phenomenon is rarely observed when the cinders come within the definition and specification of this paper.

Any concrete, however, is subject to shrinkage movements due to the hydration of the cement and to drying. Changes in moisture content cause changes in volume. Reference is here made to the paper by W. D. M. Allan, in this Journal (April 1930, Proceedings, Vol. 26, p. 699). It reports on sand concrete as well as cinder concrete. Sand-cinder concrete is perhaps intermediate in its behavior.

Part of the conclusions are quoted:

The use of units which are only partially dry, containing 2 to 5 per cent of moisture by weight, will result in the major amount of shrinkage which will occur in a concrete masonry wall.

For the conditions of air curing used in these studies, 26 days of air curing was generally sufficient to obtain blocks that would result in the minimum amount of shrinkage when laid in the wall.

In his foregoing discussion of manufacture of cinder units, the writer pointed out that the units must be piled in such a manner

that drying is facilitated, and that they must be protected against the weather.

These precautions do not apply to sand-cinder concrete, poured in situ. When properly reinforced it is unquestionably less subject to volume changes, or the evidence of such change does not become apparent. The design of the mixture is presumably of particular importance, but the data on hand do not justify any conclusions.

Fire Resistance

There is considerable evidence, and a great deal of opinion, available regarding the fire resistance of cinder and sand-cinder concrete. The majority seems to favor the material as being quite remarkable in this respect. A dissenting opinion, however, was voiced by the A. C. I. Committee E-4, in the 1929 Proceedings of the Institute (Vol. 25, p. 812). In summarizing its report, the committee concludes with the statement:

> Cinder concrete at high temperature transmits heat rapidly and loses strength.*

This may be compared with some statements of other experienced observers. For example, Prof. Ira H. Woolson (elsewhere quoted by Committee E-4) said at the building code hearing in New York City, June 1909:

> The best fire-resisting material used in construction today is cinder concrete without any qualification.

The divergency of opinion is naturally explained by the fact that the material is not clearly defined. We speak of the fire resistance of concrete—yet no two concretes behave alike. Cinders as an aggregate are less clearly defined than others—it is natural that the resulting concrete will not be uniform to the second decimal.

When cinders are improperly used (poorly graded aggregate, careless mixing and making) the resulting concrete may be so open-textured that it permits free passage of air. The heat transfer through such concrete is naturally rapid, and it is quite possible that exposed carbon particles in the interior may burn.

As an example of such poor cinder concrete may perhaps be cited the three specimens of Technologic Paper No. 130 of the

*It is unfortunate that some qualification and elucidation of this statement by Committee E-4, did not have equal prominence with the original report. It is in statements by N. D. Mitchell, A. C. I. *News Letter*, Nov. 1929, p. 11.—Editor

Bureau of Standards (referred to by Committee E-4). The cinder concretes (sand-cinder) of that paper are described as follows:

Cinder concretes Nos. 1291 and 1292 were extremely weak and friable. The appearance of the concrete indicated too great a proportion of fine material.

Cinder concrete No. 1291a was an extremely rough specimen because of a deficiency of fine material.

Such mixtures are not typical of the cinder and sand-cinder concrete of today and were evidently prepared by someone who was not familiar with designs involving this type of aggregate.

In well designed cinder concretes the fire resistance has been quite remarkable. Tests have been made at the Underwriters' Laboratories by the cinder unit industry and have shown a resistance to fire and water, and a retardance of heat transfer greater than in similar sand concrete units. As previously mentioned, the presence of coal and coke does not materially lower the fire-resistance, within limits. It lowers the strength of the concrete after fire exposure, but even a unit of pure coal and coke aggregate was not destroyed in a test reaching its temperature endpoint (8-in. hollow wall) in 3 hours and 49½ minutes.

In sand-cinder concrete, the evidence of the test house of Columbia University is of particular interest. The permanent walls of this house are built of 1:2:5 sand-cinder concrete. Professor Woolson cited this house at the 1909 hearing on the New York building code:

For these test buildings we have tried brick many times. Brick lined with cinder concrete 3 or 4 inches thick and cinder concrete buildings we have also tried, and as a result of having built several of these buildings I have arrived at the conclusion that the only and best material to use for a building of that character is the cinder concrete.

Before the convention of the National Association of Cement Users, 1911 (Vol. 11), Professor Woolson stated of this test structure, which had then been subjected, in the aggregate, to 24 hours of firing at an average temperature of 1700 degrees, with water applied at the end of tests:

Today (Dec. 13, 1910) the building is standing in as perfect condition as it was the day it was built, having some few temperature cracks which, of course, are due to that very excessively severe treatment.

Concerning this building, Rudolph P. Miller stated in 1925 (in report to Crozier-Straub, Inc.):

Since that statement was made the same structure has passed through over forty hours of such severe treatment in good condition, without the need of repairs.

Perrine and Strehan open the discussion of fire resistance of sand-cinder concrete by saying:

To any one at all familiar with the effect on cinder concrete of flames at high temperature followed by the application of cold water at high pressure, little other evidence is necessary to prove its efficiency under this treatment.

Inasmuch as Committee E-4 is chiefly concerned with the heat transfer through the material, it may be well to point to the tests by Humphrey, as reported in Bulletin 370 of the U. S. Geological Survey. If the writer reads the graphs correctly, 8-in. solid concrete blocks in similar fire tests showed about the following:

	Unexposed Face of Wall	
	Initial Temperature	After 2 Hours
8-in. granite concrete	18 deg. C.	58 deg. C.
8-in. gravel concrete	23 deg. C.	56 deg. C.
8-in. sand cinder concrete. 1:2:4, with 24.5 per cent unconsumed carbon	28 deg. C.	43 deg. C.

Weighing all the evidence at hand, both cinder and sand-cinder concrete appear to possess a remarkable degree of fire-resistance—provided they are well made. 8-in. hollow cinder blocks, 1¾-in. web, not more than 35 per cent combustible, 1 cement to 7 dry rodded cinders, are given a 3-hour rating by the Underwriters' Laboratories, and are certified as such. Cinder concrete building units in fire tests, and in actual fire exposures, have shown a remarkable absence of cracking and spalling, both when exposed to fire and to water. The status of sand-cinder concrete is not as clearly defined—but the fire record of this material has been truly remarkable.

Heat Transmission

The 1930 Guide of the American Society of Heating and Ventilating Engineers lists the following heat transmission coefficients for "cinder concrete" (without specifying the composition of the material):

Solid cinder concrete. Conductivity "k" (per 1 in. thickness) 3.33

12-in. hollow cinder block, four cores, three longitudinal webs. Conductance "C" (face to face) 0.28

8-in. hollow cinder block, two cores. Conductance "C" (face to face) ... 0.45

These values check well with those obtained in laboratory tests witnessed by the writer. The tests are summarized as follows:

At the University of Toronto, Prof. R. W. Angus[17a] conducted tests for Canadian Cincrete, Ltd. The cinder units were built into a wall, 7 blocks high and 3 blocks wide, having a total area of 20.31 sq. ft. The wall was set up in a large box in such a manner as to divide the box into two compartments. One of these compartments communicated directly with the cold room of a refrigerating machine, while the other compartment was heated electrically. Fans distributed the air over the surface of the wall.

The air on the warm side was maintained at approximately 70 deg. F. while the air on the cold side was maintained at about 20 deg. F. When the temperatures on each side remained constant, all the heat put into the warm compartment was transferred through the wall. From this the conductance and transmittance may be calculated, when knowing the temperatures of the air and the surfaces of the wall.

The units under test were 8 x 8 x 16-in. hollow cinder block, having three oval cores and approximately 40 per cent air space. The design of mix is not definitely known, but the units were made from railroad cinders and designed to meet the code requirement of 1,000 lbs. per sq. in. This should represent a relatively dense cinder concrete.

8-in. hollow cinder block wall exposed

Conductance "C" (surface to surface)	0.423
	0.415
Transmittance "U" (air to air)	0.340
	0.335

At Penn State College, Prof. F. G. Hechler[17a] conducted tests for National Building Units Corp. Two identical walls were built of the cinder units under test, each 7 units high and 3 units wide. A hot plate was fitted tightly between the two walls which were clamped together. The hot plate was surrounded by a guard ring which was maintained independently at the same temperature as the plate, so that all heat flowing from the plate passed through the two walls. The hot plate was electrically heated and the heat input as well as the temperature was known. Knowing the temperature of the outside surfaces of the walls, the conductance "C" may be computed.

8 x 8 x 16-in. hollow cinder units were tested. The units were made from railroad cinders, mixed 1:7, to pass a strength requirement of 700 lbs. per sq. in. They had two cores, or about 33 per cent air space.

8-in. hollow cinder block wall exposed

Conductance "C" (surface to surface)	0.457

This value was determined at a mean temperature of 62.7 deg. F. As the temperature was increased, the conductance increased. At 79.8 deg. F. it became .489. This must be caused by increased radiation and convection through the hollow space.

It was also found that the conductance was less when the flue formed by the air space of the hollow units was sealed at the top than when it remained open. The convection of the hollow space evidently increased when the flue acted as a chimney.

4 x 8 x 16-in. solid cinder units, representing the same type of cinder concrete, were also tested, laid in walls of the same size.

4-in. solid cinder concrete exposed
Conductance "C" (surface to surface)
Mean temperature 56.3 deg. F.....1.005
Mean temperature 68.8 deg. F.....1.04
Mean temperature 85.3 deg. F.....1.06

The Conductivity "k" per 1 in. thickness of this solid wall may be figured as 4 x 1.00 = 4.00 (4.25 for the highest temperature). This value is higher than that of 3.33 used in A. S. H. V. E. Guide—but it must be remembered that the 4-in. wall contained mortar joints, comprising probably 10 to 15 per cent of the wall area. If we accept the value of 8.3 (generally used) for the conductivity of this mortar, the value of 3.33 for the conductivity of the cinder concrete checks well with the test.

It may not be amiss to point out that considerable confusion prevails in many discussions of heat transmission data. The nomenclature is not uniform, and it is not always made clear to what conditions the coefficients reported belong. It is quite common, for example, to apply data obtained on one small unit to larger wall assemblies of that unit, or to compute the coefficient for a hollow wall on the basis of data for the solid material without taking into account the air space. In the present paper, the writer is adopting the nomenclature of the A. S. H. V. E.

As far as sand-cinder concrete is concerned, Prof. C. L. Norton reports on a 1:2:4 mixture, tested by the hotplate method. Without giving details regarding the make-up of the concrete, Norton states the conductivity as follows:

1:2:4 sand cinder concrete
Conductivity "k" (per 1 in. thickness)
Mean temperature 122 Deg. F.....2.35

On the other hand, the Guide of the A. S. H. V. E. quotes Peebles as follows:

(Sand-) cinder concrete (110 lbs. per cu. ft.).
Conductivity "k" (per 1 in. thickness)
Mean temperature 75 deg. F.....5.20

These two values are far apart. Aside from probable differences in method (the one used by Peebles is not stated), it is possible that the concretes may have been quite different in their make-up. Norton reports the conductivity for 1:2:5 stone concrete to be 6.27, while Peebles reports the value for stone concrete weighing 145 lbs. per cu. ft. as 6.30. Willard, Lichty and Harding report the value for 1:2:4 stone concrete as 8.30 (140 lbs. per cu. ft., mean temperature 110 deg. F.), and this is the value used in the calculations of the A. S. H. V. E. Guide. It would appear that the value reported by Norton is too low for the average product—perhaps the value reported by Peebles is too high. The method employed and the nature of the material must be known in detail if one is to judge the significance of heat transmission coefficients.

The heat flow through a wall depends to a great extent upon the pore structure of the wall material. If the pores are large and open, free passage of air may occur, and the heat transfer is considerable. The writer knows of cinder concrete, molded on a primitive hand press, which was so porous that a 30-mile wind could be felt through an 8-in. wall of the material. This, however, is not the type of concrete here considered. Following the recommendations for grading, mixing and making, here proposed, the resulting concrete will have the porous, cellular particles of cinders embedded in a relatively dense matrix; the cells will contain "dead air," and contribute to the insulative quality of the material.

The addition of sand will affect the pore structure. There are no data available to show to what extent. Such data can only be produced by carefully designing the concrete mixtures and keeping all variables of test under control.

The pore structure of the concrete is of particular importance when fire retardance is concerned. An open-textured cinder concrete will not only permit a rapid transfer of heat, but may also permit the fire to affect the interior of the concrete. When discussing fire resistance, it was pointed out that such a concrete might well justify the opinion which certain authors hold concerning cinder concrete, but such concrete is not common.

Sound Absorption

The problem of reducing sound reverberation in large rooms, and sound transmission through partition walls is involved. Some experimental data are available regarding cinder concrete building units, but these data will not be reviewed here. Reference is made to manufacturer's publication.

The writer has observed, on numerous occasions, the marked sound absorption of cinder concrete, either exposed or painted. In one building particularly where some of the cinder unit walls were painted while others were plastered, the difference in sound reverberation between rooms of different wall finishes was immediately noticeable. The texture of the cinder concrete—open, when compared to that of ordinary plaster—should be of value in improving the acoustics of the room.

The transfer of sound through a material is naturally considerable when the pore structure is such as to permit free passage of air. Plastered partitions of cinder concrete appear to have given a high degree of satisfaction in many jobs like hotels, hospitals or apartment buildings where sound insulation is an important matter. The writer bases this statement on personal observations, rather than on quantitative tests.

There are apparently no data available regarding the behavior of sand-cinder concrete in this respect.

The Material Generally

Cinder concrete, properly made, will receive and hold nails to an extent depending upon the nature of the cinders, the design of mix, the method of making and the age of the concrete. The advantage of nailable concrete is particularly obvious in walls where the direct fastening of furring strips, plastering grounds, and the like, results in marked economies of construction.

Nailability depends upon the nature of the aggregate, cellular anthracite cinders here being preferable to denser, well clinkered bituminous cinders. Such other factors as cement/aggregate proportion, amount of mixing water, degree of compacting of concrete, naturally affect nailability as they affect the pore structure of the material. The fact remains, however, that cinder concrete building units as usually manufactured (proportions 1:7 to 1:10) to meet a 700 lbs. requirement, are readily nailable

at the time when they are usually placed in the wall. Numerous tests have been made on the holding power of such nails, but the tests have not been devised to show the relation of nailability to the other characteristics of cinder concrete. Addition of sand to the aggregate naturally reduces nailability; quantitative data are not available.

Machine-made cinder concrete building units, made from a relatively dry, non-plastic mix, have an interesting surface texture, caused by the roughness of the particles of aggregate and developed through the method of moulding. Unusual architectural effects have been obtained by painting this surface without destroying the texture. The adhesion of plaster, forced into the crevices of the surface, appears to be considerable; demonstrations of the strength of mortar bond have been made, but no accurate test data are available. The surface texture of wet cast, denser sand-cinder concrete is less interesting.

History and Bibliography

It is not within the scope of this paper to deal with the history of the use of cinders in concrete, beyond the incidental mention of certain facts already considered. It is only just, however, to point out two developments which have greatly influenced the status of the material in the United States.

First, as to sand-cinder concrete. The recognition given to this material in the City of New York, through the work of Ira H. Woolson, Rudolph P. Miller, A. L. A. Himmelwright, Harold Perrine and George E. Strehan, is an important factor in its development, often in the face of the strongest opposition. Fifteen years ago, sand-cinder concrete was a very live subject of discussions—for some unknown reason it lost its actuality.

Second, as to cinder concrete building units. The work of F. J. Straub in putting his "cinder block patent" into practical use is to a great extent responsible for the development of a cinder unit industry. It has been the subject of much litigation—and the writer has been careful to avoid, in this report, all controversial issues. These are reserved for other forums than that of the A. C. I. But the writer wishes to acknowledge his debt to F. J. Straub, and his associates in the present industry, who had the vision and the common sense necessary to overcome a multitude of prejudices.

In the following bibliography, the writer has listed only the papers which he considers to be of primarly interest to the student of the subject. So much of the published data are merely repetitions, and a complete bibliography, including every publication in which cinders are mentioned, would fill many pages and would be of little value.

1. Charles L. Norton—*Engineering News*, October 1902; *Engineering News*, January 1904. Tests to determine the corrosion of steel in concrete.

2. R. T. Surtees—"A few tests and experiments with reinforced concrete." *Engineering Record*, December 14, 1907. Tests to determine the corrosion of steel.

3. Richard L. Humphrey—"The strength of reinforced concrete beams." U. S. Geological Survey, Bulletin 344, 1908.

4. R. L. Humphrey and L. H. Losse—"The strength of reinforced concrete beams." U. S. Bureau of Standards, Technologic paper No. 2, 1912. Two papers reporting interesting data on the design of sand-cinder concrete mixtures.

5. D. B. Butler—"The dangers of breeze concrete." The Journal of the Society of Architects, April 22, 1909. Expansional movement.

6. Ira H. Woolson. *Cement Age*, October 1909. Testimony before the New York Building Code Commission, summarizing experience with sand-cinder concrete in fire tests. Numerous reports by Woolson on fire tests involving sand-cinder concrete will be found in various technical journals, but are not listed here.

7. Charles L. Norton—"Some thermal properties of Concrete." Proceedings, National Association of Cement Users, 1911, p. 78. Heat transmission of sand-cinder concrete. Fire resistance discussed.

8. Ira H. Woolson and Rudolph P. Miller—"Fire tests of floors in the United States." Proceedings, International Association for Testing Materials, Paper XXVII, 2nd. section, 1912. Report on a large number of fire tests involving sand-cinder concrete floors of different designs.

9. Guy B. Waite—"Cinder Concrete Floors." Transactions, American Society of Civil Engineers, Vol. LXXVII, p. 1773, 1914. Tests on reinforced sand-cinder concrete floors.

9A. M. C. Tuttle—"Bibliography of Corrosion of Iron and Steel in Cinder Concrete." *Engineering News*, April 18, 1912.

10. Harold Perrine and George E. Strehan—"Cinder concrete floor construction between steel beams." Transactions, American Society of Civil Engineers, Vol. LXXIX, p. 523, 1915. Tests on reinforced sand-cinder concrete floors, presenting a review of the state of the art, and followed by interesting discussion.

11. H. Nitzsche—"Untersuchungen an Kesselschlacke für Betonzwecke." *Zement*, March 1920. A comprehensive paper on cinder concrete.

12. Arthur Holmes—"Geological and Physical Characters of Concrete Aggregates." British Fire Prevention Committee, Red Book No. 256, 1921. Data on various types of cinders.

13. H. Nitzsche—"Die Bedutung der Schwefelsauren Salze der Kohlenschlacken für die Beständigkeit der Schlackebeton-Erzeugnisse und für deren Verhalt bei wechselnder Feuchtigkeit." *Zement*, 1925, p. 338. Expansional movement due to the presence of sulphur compounds. Also tests on washing of cinders.

14. F. M. Lea and F. L. Brady—"Slag, Coke Breeze and Clinker as Aggregates." Department of Scientific and Industrial Research, Building Research, Special Report No. 10, London 1927. Brief review of these aggregates.

15. F. M. Lea—"Investigations on Breeze and Clinker Aggregates." Department of Scientific and Industrial Research, Building Research, Technical Paper No. 7, London 1929. A very comprehensive study of the effect of coal and sulphur compounds upon the permanence of cinder concrete.

16. F. L. Brady—"The Corrosion of Steel by Breeze and Clinker Concretes." Department of Scientific and Industrial Research, Building Research, Special Report No. 15, London 1930. Test on corrosion of steel in contact with cinders and cinder concrete.

17. National Building Units Corporation, Philadelphia, has published considerable data relating to cinder concrete building units manufactured under the Straub patent.

a. Technical Series: Bulletin No. 1: "Sound Absorption of Cinder Concrete Building Units."

b. Technical Series: Bulletin No. 2: "Three hour fire and water test made upon walls of 8 inch and 4 inch hollow Cinder Concrete Building Units."

c. Technical Series: Bulletin No. 3: "Thermal insulation of masonry walls."

d. Report of Underwriters' Laboratories on Straub Hollow Cinder Concrete Building Blocks, Retardant No. 1429, 1922. With supplementary data revising specifications, May 14, 1930.

e. Memorandum on Cinder Concrete Building Units and their place in modern building codes.

Many data regarding cinder and sand-cinder concrete, including a very complete list of patents, will be found in the Transcript of Record in the case of Crozier-Straub et al. vs. Graham et al., United States Circuit Court of Appeals, Third Circuit, March Term 1928.

Admixtures And Workability of Concrete

BY G. M. WILLIAMS*

*Two critics who considered this paper on behalf of the Publications committee recommended that to increase its practical value the author should designate the admixtures by name, rather than disguise them with numbers. Professor Williams has graciously responded with the names and they appear in a footnote on the page with Table 1, in which the admixtures are first listed by number. In justice to the author and to producers of admixtures here given the lower ratings, the author's viewpoint, with respect to his contribution, requires emphasis. Professor Williams' brief paper is to be read as a suggestion for a different approach to the study of admixtures and not for the limited data it presents on the relative merits of the admixtures he used in his studies of a method of test. The author writes: "My purpose was to outline a method of test for the rating of admixtures rather than to discuss the merits or demerits of any particular one or group * * * A satisfactory test should be established first * * * Had I intended to compare the admixtures in this manner I would have included several others which have been used commercially, one of them approaching fairly near to No. 6 in efficiency * * * I included a minimum of test data in tabular form to make comparisons easier * * * Strength, porosity and permeability must be taken into account in any final rating of an admixture, but I felt that to discuss these factors at length would only divert attention from the test methods proposed, toward the admixtures themselves. The statement that admixtures tested 'do not appreciably affect strength or permeability when used in the amount or proportions specified by the manufacturer' briefly sums up any discussion which I might make in connection with these factors." Thus while we*

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may look forward to further data on admixtures from Professor Williams, his present contribution is aimed at the method of studying their values and not at the relative values obtained from such study.—EDITOR

IN RECENT YEARS the use of admixtures in concrete has greatly increased due to advantages which their use has demonstrated under certain conditions of concrete practice. The advantages advanced for admixtures, by the manufacturers, vary, but in general, improvement in strength, water-tightness and workability is claimed. The effect of admixtures on the first two qualities can be measured by testing procedure already standardized or developed but their effect on workability is not so easily measured owing to different views as to what is meant by workability as well as absence of well established test to measure this property.

By workability, the writer means the ability of a mixture to remain homogeneous and free from segregation during the process of transportation and placement. Workability is a relative condition. A concrete might have satisfactory workability under certain conditions but be unworkable or segregate badly under others. While this definition may not agree with various laboratory definitions of the term workability, it is believed that the statement defines workability from the view point of the practical concrete maker. In practice a segregating or poor working mixture may result, after transportation difficulties have been overcome, in stone pockets, laitance, porosity, excessive permeability and low compressive strength as well as increase the cost of handling and placing if properly attempted. A homogeneous, workable mix will after placement be free from the foregoing visible and hidden defects. The practical worker will refer to such a mix as "workable."

Non-workability or segregation may be due to low mortar content, an excessively wet or soupy mortar, an excess of coarse aggregate or to a combination of these characteristics. Low mortar content may be due to low cement content (lean mix) with an aggregate which may be well balanced with respect to relative quantities of fine and coarse aggregate. The use of

additional mixing water to obtain flowability as well as workability results, in turn, in an excessively wet segregating mortar which flows away from the coarse aggregate or which permits the coarse aggregate to settle out towards the bottom of the container. Fairly rich mixtures of low sand-gravel ratio may also lack a sufficient volume of mortar to carry the volume of coarse aggregate, and the attempt to attain sufficient flowability to give easy placement results in segregation. The flowability of a mass of concrete is due entirely to the mortar content and when this is insufficient in quantity or too wet to adhere properly to the coarse aggregate particles, segregation will result.

A non-segregating homogeneous mix must have a sufficient volume of mortar to float and hold in suspension the coarser aggregate particles. This implies not only a relatively large mortar volume but a sticky plastic mortar which will adhere to and carry along the aggregate particles without segregation when the mass is set in motion.

For concrete made with given materials workability may be improved by increasing the volume of mortar which in turn will permit the use of a less fluid, more adhesive mortar without reducing the flowability of the whole. The addition of any material to the mix which will increase the relative volume of the mortar will therefore improve workability and that material which for a given weight will produce the greatest mortar volume will probably be most efficient providing the other properties of strength and watertightness are not injured.

The writer has tested a number of admixtures, including portland cement itself as an admixture, and believes that the following conclusions with respect to their effect on workability are justified.

1. All admixtures tested, including portland cement as an admixture, increased relative mortar volume, tended to reduce segregation, and showed merit as a means of increasing workability although not to the same degree.

2. The relative efficiency of admixtures in increasing workability (reducing or preventing segregation) is measured by the relative volumes of paste produced when equal weights to admixtures are made into pastes of approximately the same flowabilities.

Six admixtures of Canadian and American manufacture, in-

cluding portland cement were tested. They were all of the practically insoluble, finely divided powder form, and ranged in composition from silica and lime to clay in the form of bentonite. When prepared in neat paste form with water they varied considerably in adhesiveness or stickiness, especially with low water contents, but much of the stickiness was lost when sufficient water was added to bring them to the approximate consistency which they would have as a part of the mortar volume of a workable concrete mixture.

The following results will indicate the test methods employed to furnish the conclusions outlined above.

TABLE 1—RELATIVE VOLUMES OF PASTES OF SAME FLOWABILITY PRODUCED WITH EQUAL WEIGHT OF ADMIXTURES

Admixture*	Relative Amount of Mixing Water Required	Relative Volume of Paste Per 100 Grams of Admixture
1 (cement)	1.0	1.00
2	3.2	1.95
3	3.3	1.95
4	2.4	1.62
5	3.4	2.00
6	13.0	5.70

In Table 1 it is seen that all admixtures were more efficient than cement as measured by volume of paste produced. Four of the admixtures are grouped fairly closely while No. 6 falls in a third group.

Equal weight of admixtures were then added to identical cement-sand mortars and the volume increase due to the presence of the admixture was determined as listed in Table 2.

TABLE 2—RELATIVE VOLUME INCREASE PRODUCED IN IDENTICAL CEMENT-SAND MORTARS TO WHICH EQUAL WEIGHTS OF ADMIXTURES WERE ADDED. ALL MORTARS WERE OF SAME FLOWABILITY AFTER ADMIXTURES WERE ADDED

Admixture*	Relative Volume of Mortar	Relative Increase in Mortar Volume Per 100 Grams Admixture
None (Plain Mortar)	1.000	——
1 (Cement)	1.040	1.0
2	1.045	1.1
3	1.045	1.1
4	1.052	1.3
5	1.060	1.5
6	1.165	4.7

*No. 1 = Canada Portland Cement; No. 2 = Vankelad A; No. 3 = Vankelad B; No. 4 = Barnsdall Admixture; No. 5 = Hydrated Lime; No. 6 = Celite. See Editor's note at beginning of this paper.—EDITOR

Comparing the results in the right hand column of Table 2 with the same column of Table 1 it is apparent that the relative volume increases in these particular mortars were not of the same magnitude as were those found for the relative volumes of neat pastes but the values are somewhat proportional. Apparently some of the paste has gone to fill void space and the rest swells the volume of the mortar.

Tests were then made to measure the actual segregation which occurred in identical concrete mixtures to which were added equal weights of the different admixtures. Flowability, as measured by spread of mortar on the flow table was made constant by the addition of sufficient water in each case. The mortar content of the batch was not excessively wet, but rather sticky and plastic, and segregation was obtained by the use of an excess of ¾ in. to ⅜ in. aggregate. Amount of segregation was measured by weighing the coarse aggregate particles which rolled from the mass beyond the mortar line on the flow table. It was difficult to obtain concordant results in repeated tests of any one batch or in tests of successive batches since it was difficult to select and properly place a fair sample of the batch in the flow table mold, all of the batches segregating to varying degrees. The results shown in Table 3 do indicate much and varying segregation, but do not permit of rating the admixtures, which in the other tests are shown to be of somewhat equal merit.

TABLE 3—PERCENTAGE OF COARSE AGGREGATE SEGREGATED FROM CONCRETE MASS AND PERCENTAGE OF MORTAR IN CONCRETE

Admixture*	Ratio Volume Mortar to Volume of Concrete	Percentage (by weight) of Coarse Aggregate Segregated from Mass
None (Plain Concrete)	.472	13.13
1 (Cement)	.479	11.30
2	.482	9.91
3	.482	10.57
4	.482	9.90
5	.487	9.92
6	.514	4.95

Since it is difficult to handle and sub-divide a segregating concrete the values above in Table 3 should be considered only approximate, but they indicate greatest segregation for the plain concrete, slightly less for cement as an admixture, still less of the group of four admixtures and least for admixture No. 6.

To determine definitely whether or not the value of an admixture in promoting workability is proportional to its relative paste volume concretes similar to those included in Table 3 were prepared, but instead of using equal weights of admixtures the amounts were increased so as to make the paste volumes of admixtures equal. From the results of Table 1 the relative weights of admixtures required were found to be as follows:

Admixture*	Relative Weights of Admixtures to Furnish Equal Volumes of Admixture Paste
1 (Cement)	5.7
2	3.0
3	3.0
4	3 5
5	2.8
6	1.0

TABLE 4—PERCENTAGE OF COARSE AGGREGATE SEGREGATED FROM CONCRETE MASS AND PERCENTAGE OF MORTAR IN CONCRETES

Admixtures Added to Furnish Equal Volumes of Admixture Paste

Admixture*	Relative Weight of Admixture Used	Ratio Volume Mortar to Volume of Concrete	Percentage by Weight of Coarse Aggregate Segregating from Concrete
1 (Cement)	5.7	.517	4.20
2	3.0	.510	4.85
3	3.0	.514	4.95
4	3.5	.514	4.73
5	2.8	.518	5.00
6	1.0	.517	3.98

Results of tests in Table 4 indicate that the additions of admixtures in quantities to produce equal volumes of admixture pastes do furnish concretes of practically equal mortar contents and equal workability as measured by segregation of coarse aggregate.

From the foregoing test results it would appear that the mortar content of a batch, for these particular aggregates, should be well in excess of 50 per cent to insure workability, and for concretes which may be subjected to considerable jarring and shaking during transportation not only a relatively large volume of mortar but in addition a mortar of comparatively low flowability would be required.

While admixtures are relatively more efficient than cement in obtaining a concrete of workability suited to any given condition

of transportation and placement the writer does not suggest that their use is always necessary or advisable. For concrete of any required cement content first attention should be directed to the most desirable sand-gravel ratio. Relative increase in sand content increases mortar volume which in turn permits the use of a less fluid mortar, thereby increasing workability and postponing likelihood of segregation. After adjusting the sand-gravel ratio and finding segregation under the conditions of use, or that the mix is near the segregation limit the use of an admixture will be advantageous. In work where a surface as true as the face of the mold is desired or in any case where the available fine aggregate is coarse and harsh, lacking in fine particles, the use of an admixture may be of value in obtaining satisfactory results.

The factors of strength and watertightness should not be forgotten. Generally these properties cannot be sacrificed to obtain workability. Tests which the writer has made quite definitely indicate that admixtures do not appreciably affect strength or permeability when used in the amounts or proportions specified by the manufacturer. In cases where excess quantities of an admixture are needed to produce satisfactory workability the possibility of reduction in strength and watertightness must receive consideration. On the basis of workability alone, the data in Table 1 would indicate the use of an admixture to be dependent mainly upon the relative cost of cements and admixtures. The use of admixtures 2 to 5 would not be economical unless their cost were less than twice, and No. 6 less than five or six times, the cost of portland cement. Consideration should also be given to the fact that cement not only increases workability as an admixture but at the same time it definitely increases strength and impermeability.

Readers are referred to the JOURNAL *for May 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931. (Normally the discussion of a paper in the February* JOURNAL *would appear in June but is in this instance scheduled for May to combine it with discussions of "Slump Tests and Flow-Table Determinations"—two papers appearing in the January* JOURNAL, *involving in part a different approach to the same fundamental problem.)*

A Comparison of Continuous With Batch Mixers In Plant Operation*

BY BENJAMIN WILK†

INTRODUCTION

The advantages of a batch mixer as against a continuous mixer have been the subject of considerable discussion among concrete products manufacturers for many years. The batch mixer has generally been favored because of its longer mixing time and the simplicity with which aggregates can be proportioned into the mixer. The continuous mixer has had its advocates because of the greater amount of concrete that could be mixed per day per dollar of investment.

Until recent years continuous mixers were built generally with short mixing troughs, many of them not over three feet long and with uncertain feeding devices. Mixing time was therefore very short, frequently less than one-half minute. This type of continuous mixer has not been very widely used. Improvements in design, however, during recent years. have developed continuous mixers with more accurate feeding devices and longer mixing troughs.

To determine the comparative value of the improved continuous mixers as against batch mixers a series of tests was made in the plant of the Standard Building Products Co., Detroit, Mich., in 1929 and 1930. Batch mixers had been used in this plant for several years, but a machine requiring a continuous, large supply of concrete was being installed and the question arose whether to use a continuous mixer of a moderate size, or a very large batch mixer. To obtain a practical comparison a continuous mixer was installed. Tests were carried on under the

*For presentation and discussion at the 27th Annual Convention.

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supervision of the writer, assisted by J. Paul Wilson, engineer of the Concrete Products Association of Detroit.

TEST PROCEDURE

Comparative tests were made over a period of six months at intervals of approximately 30 days commencing in November, 1929, and ending in May, 1930. Each test consisted of two successive batches made in the batch mixer followed by an equivalent amount in the continuous mixer. Control was obtained by measuring aggregates in a one cubic foot box for the batch mixer and by determining the proper setting of feeding devices on the continuous mixer based on the yield in terms of block per sack of cement for the previous day's run.

Each batch consisted of three wheelbarrows of aggregate and one sack of cement. Each wheelbarrow contained exactly three cubic feet of aggregate. Batches were mixed three minutes dry and three minutes wet. This mixing procedure was used for the batch mixer because it was found to have given practically the best results in a series of tests made by the Committee on Plant Operation of the American Concrete Institute, reported in the 1929 Proceedings. The amount of water was just enough to give a water-web mark on the finished product. The mixed concrete dropped directly into a hopper feeding the block machine. The yield from these two batches was 39 block, or an average of 19½-eight in. block per sack of cement.

Aggregate and cement were fed simultaneously through the continuous mixer. Two sacks of cement were put into the cement hopper and a measured flow of aggregate started into the aggregate hopper. The mixer was allowed to run until all of the cement from the two sacks ran out. The aggregate and cement was mixed dry in the mixing trough for a distance of approximately thirty-two inches, or one-third the length of the trough. At this point water was added in just sufficient quantity to give a water-web mark on the finished product. Mixing time in the continuous mixer was determined by dropping color in with the cement and also by small colored pieces of wood. Mixing time was slightly less than one minute. As the mixed concrete dropped off the end of the trough it was shoveled into wheelbarrows and immediately moved to the hopper above the block machine. The

yield from the continuous mixer varied from 38 to 43 eight in. block per two sacks of cement, or an average of 20¼ block per sack. The machine in which the block were made was directly below the batch mixer. The block from both mixers were made by an experienced operator in the usual plant practice. They were placed on two racks set in the curing room, one behind the other, so that all the block had the same curing conditions.

AGGREGATE

The aggregate used was washed sand and pebbles so combined as to produce a fineness modulus of 3.4. Sieve analyses of the combined aggregate made at intervals, checked very closely with the calculated finess modulus.

DESCRIPTION OF MIXERS

The continuous mixer was an Eberling driven by a 5 h. p. motor. The cement hopper had a capacity of slightly more than two sacks of cement, and the divided hopper for the aggregate held slightly more than four cubic feet. A chute ran directly from an overhead bin to the aggregate hopper, so that there was a continuous flow of aggregate into the hopper. The feeding belts under the hoppers were 18 in. wide. The roller under the cement hopper was shaped as an octagon. This caused a slight movement of the belt as it passed under the cement hopper and thereby eliminated any arching that might have developed. The mixing trough was semi-circular with a diameter of 20 in. and an overall length including dry and wet mixing of 9 ft. There were 34 double blades, each one approximately 17 in. long, revolving at a speed of 35 r. p. m., mounted on a 3-in. shaft. Theoretically each particle of concrete was pushed ahead and therefore mixed 68 times in traveling through the mixing trough. The volume of concrete mixed per minute was approximately 4 cu. ft.

The batch mixer was a standard oversize 9 cu. ft. Blystone side dumping mixer equipped with six blades. A 15-gal. water tank directly above the mixer was used to control the amount of water added per batch.

CURING AND TESTING OF BLOCK

The block were cured for twenty-four hours in steam at a temperature of approximately 90 deg., and then cured in air

until 72 hours before time of testing. They were then delivered to the laboratory of the Concrete Products Association of Detroit, and stored inside at room temperature for the remaining 72 hours before they were broken. Test results are shown in the following table. "C" results are from the continuous mixer, "B" results from the batch mixer.

SEVEN DAY TESTS

Series No.	C-1	B-1	C-2	B-2	C-3	B-3	C-4	B-4	C-5	B-5
Yield Per 2 Sacks	38	40	42	39	40	39	43	39	41	39
	770	690	680	740	560	580	740	820	560	660
	830	730	640	770	530	580	710	790	610	640
	820	710	610	700	550	600	690	790	630	580
	807	710	643	736	547	587	713	800	600	627

FOURTEEN DAY TESTS

Series No.	C-1	B-1	C-2	B-2	C-3	B-3	C-4	B-4	C-5	B-5
Yield Per 2 Sacks	38	40	42	39	40	39	43	39	41	39
	980	1060	840	1010	780	750	510	910	750	600
	1060	770	840	1000	810	790	530	930	630	620
	1020	915	840	1005	795	770	520	920	690	610

The following table shows comparisons after results were computed on the basis of 39 block per two sacks.

SEVEN DAY TESTS			FOURTEEN DAY TESTS		
Series No.	Yield 2 Sacks	Av. of 3 Block	Series No.	Yield 2 Sacks	Av. of 2 Block
C-1	39	780	C-1	39	990
B-1	39	740	B-1	39	940
C-2	39	690	C-2	39	910
B-2	39	740	B-2	39	1000
C-3	39	560	C-3	39	810
B-3	39	590	B-3	39	770
C-4	39	790	C-4	39	570
B-4	39	800	B-4	39	920
C-5	39	630	C-5	39	730
B-5	39	630	B-5	39	610

The grand average for the seven day block from the continuous mixer is 690 lb., from the batch mixer 700 lbs. The grand average for the fourteen day block from the continuous mixer (including

the unusually low figure of 574 lb. in Series 4) is 802 lbs., and from the batch mixer 848 lbs.

The final testing was at fourteen days because in the Detroit market it is desirable to be able to deliver block on the job at the end of fourteen days which have a strength above the 700 lbs. requirement.

CONCLUSION

The test results indicate that the variation between the strength of the block made from the continuous mixer of the long trough type and the usual batch mixer is very small, although the time of mixing in the continuous mixer is approximately one minute as against six minutes mixing in the batch mixer. Evidently the concrete in this type of continuous mixer is given a thorough mixing by the large number of paddles and the large number of revolutions per minute.

Readers are referred to the JOURNAL *for June 1931 for discussion which may develop. Such discnssion should reach the Secretary by May 1, 1931.*

Discussion of Report of Committee 201

"Blast Furnace Slag as Concrete Aggregate"*

BY L. W. WALTER†

The completeness of this report, dealing as it does with the processing of slag, its physical and chemical properties and with the properties of slag concrete, entitles it to a rating as probably the best single contribution to concrete literature ever presented on this subject.

This may well be true, as the time was opportune, and the National Slag Association had made available to interested committees engaged in formulating specifications for concrete and concrete aggregates a vast amount of information covering test data and service history of slag and slag concrete. This information, furnished through symposiums issued at regular intervals over the last two years, covers more than 500 pages and represents, as far as I can recall, the most comprehensive collection of data on any single concrete aggregate.

This brings to my mind the very meager published information obtainable on slag and slag concrete when, early in 1914, I attempted a collection of available data preliminary to a survey of concrete structures in which slag had been used as a coarse aggregate. At that time, if you wanted real information about slag and needed it, you had to go into the field and get it. I had previously known of its use by iron and steel companies to the exclusion of other aggregates. I had interviewed their engineers, had observed their methods, and examined the materials used. These included natural, puzzolan, and portland cements, granulated slag as a substitute for sand, and the use of coarse aggregate ranging from bank-run of air-cooled slag to pancake slag.

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The pancake slag was sufficiently cooled with a water spray to produce brittleness which would encourage its breaking up when dropped into steel gondola cars. The cars, after loading, were shifted to the concrete mixer and the slag used in cold weather before the initial heat had passed off—in some cases from the blast furnace into the concrete on the same working day.

My survey in 1914 of slag concrete structures was largely but not entirely confined to structures at iron and steel plants in the Cleveland, Youngstown, Pittsburg, Johnstown and Birmingham districts. Several Erie railroad structures in which slag had been used as coarse aggregate were carefully inspected, and conclusions then and since have been of a nature favorable to the continued use of slag. As an indication of my viewpoint after this survey, I quote from my discussion of a paper on "Blast Furnace Slag as an Aggregate in Concrete," presented by W. A. Aiken at the A. S. T. M. convention in June, 1914 (A. S. T. M. Proceed., Vol. XIV):

> I desire, therefore, to present for your consideration two reasons why the use of slag as a material in concrete has been so limited, and endeavor thereby to encourage discussion of a subject which must be dealt with in the concrete industry of the future.
>
> 1. Among these reasons must be recognized a lack of confidence in slag as a concrete aggregate, due rather to a limited individual experience with its use and to a very limited publication of data by those who have used it, than to specific objection to it based on personal experience with it, or on a knowledge of its behavior where used by others.
>
> 2. There exists a quite prevalent idea that slag contains elements which may act detrimentally when in combination with cement.
>
> Let us see to what extent these are factors. They are certainly not factors as affecting construction and maintenance of masonry at the large iron and steel plants, where slag has for several years been used to the exclusion of other coarse aggregate in concrete. Here the use of slag was first limited to structures, failure in which would not be vital to the operation of the plants, but confidence once established seems to have begat confidence, and as a result there may be seen at most steel plants, slag concrete in all classes of masonry, including foundations subjected to the pounding of heavy machinery, the stoppage of which, but for an hour, might entail a loss compared with which the first cost of the foundation would be insignificant. I know of no higher compliment that can be paid to slag as a concrete aggregate than its continued use in mill construction by those who have used it most, and who are in position to know it best. This, however, should not be accepted as an argument in favor of its use in concrete for all purposes other than mill construction.

Let us make inquiry as to what is known of slag as a concrete aggregate by those who have not been associated with iron and steel mill construction. Ask of the average man interested in the concrete industry, "What do you know of slag as an aggregate in concrete?" "Have you ever used it?" "What do you think of it?" Then figure on a percentage basis and you will likely conclude that the limited use of slag in concrete for general purposes is due in part to a lack of knowledge as to its behavior where used, and to a suspicion rather than to a knowledge on the part of some that slag as an aggregate will encourage disintegration in concrete.

Each has its bearing on past usage of slag. Each, therefore, will have its influence on future usage until there is sufficient data available to enlighten those interested as to what has been done. In order to allay all fears, these data must be of such a nature as will justify confidence based on both test results and a knowledge of the behavior of slag concrete in service for several years in the various kinds of structures and classes of work.

The revenue to be had from the utilization of slag as a waste by-product, and the desire for a material of economic advantage should stimulate the publication of reliable data concerning the use of slag in concrete, but the commercial feature must not be the predominating factor, as slag must stand on its merits. Its adaptability for use in concrete road construction, in typical highway and railroad structures, and for general building purposes, must be judged from experience with its use, separately in the different classes of work.

* * *

The sum total of all comparative test results which I have been able to collect indicates that concrete in which slag was used compares favorably in strength with that in which either limestone, trap rock, or gravel was used as a coarse aggregate. Based on a careful examination of concrete in actual structure service, I am inclined to the opinion that the failures or partial failures in slag concrete have not been proportionally greater than in concrete in which other materials of accepted good quality have been used.

I have, accordingly, been looking forward with expectancy for a period of fifteen (15) years to the ultimate publication of a report such as the Committee has presented in fulfillment of the need then expressed. In the meantime, I have taken more than a casual interest in current publications of information on slag, and have followed closely the activities in the field of specification development.

The publicity which has been given since 1914 to the use of slag has brought to light many other slag concrete structures in existence at that time. When the Ulmer Building, a reinforced concrete structure in Cleveland, was torn down in 1921 after standing fifteen (15) years, I was privileged along with several other men now members of the Institute to inspect this building

during its demolition. This one instance offered particularly convincing evidence that slag concrete will protect steel against corrosion when the covering is sufficient and the concrete is carefully placed. The inspection of this building is only one of the many observations made which, collectively, afford a background of information that justifies added confidence in the suitability of slag as a concrete aggregate.

Since 1914 my close association with concrete construction on the Erie Railroad and with technical committees has afforded an opportunity to observe the durability of new and old structures of slag concrete in a climatic zone not favorable to exposed concrete structures. I have observed nothing which indicates that any mistake was made in the use of slag.

Although there are no standard tests for durability of concrete aggregates, the sodium sulphate test and the freezing and thawing test are more generally used than any others, and seemingly give promise of ultimate standardization. As brought out on page 194-5, these tests suggest a relatively high degree of durability in slag aggregate, and this is borne out by observation of its behavior in railroad ballast, macadam roads and in sewage disposal beds. These observations, in addition to those made of its behavior in concrete, lend added confidence in its durability.

Information on methods adopted for better quality control in the processing of slag has from time to time been furnished by representatives of the producing interests who have been assigned to memberships on various committees, and slag is now commercially available which is well-graded, free from foreign materials, and handled in the loading by methods calculated to minimize segregation.

In recent years, the trend in concrete practice has been toward higher sanded mixes than were common under the old-time arbitrary proportions. More consideration has been given to the bulking of moist sand, and compensation has been made for it in measuring the sand in volumetric field mixes. The adoption of the water-cement ratio principle and the broader experience with the design of concrete mixes have led to the use of higher sanded mixtures in the interest of better workability, and these have been justified by test results. The subject matter on pages 214-

15, calculated to justify the use of higher sanded mixes, is in my opinion sound. This trend has been decidedly favorable to the extended use of slag aggregate, since it overcomes in a large measure the former objections to the harshness of slag concrete in undersanded mixes. Examination of structures exposed over a long period in freezing climates indicates that outstanding examples of excellent performance are associated generally with highly sanded mixtures.

In conclusion, the fact that the technical committees of practically all the national organizations interested in construction materials have approved slag as a concrete aggregate in their specifications and reports is the one best assurance to the engineering profession that slag is a suitable coarse aggregate for use in concrete and reinforced concrete.

BY P. J. FREEMAN.*

This report should receive more than the casual approval of the American Concrete Institute for the manner in which the Committee has handled its work, the completeness of the review and the method of clearing its findings to all other committees in this and similar organizations.

Blast furnace slag as a commercial commodity has been considered one of the newer ingredients in mass concrete and reinforced concrete construction in perhaps the larger portion of the United States. Its "newness" after one investigates briefly is not that at all. There are examples of concrete construction almost as old as the art of concrete making itself in which blast furnace slag was the large aggregate. The material is handicapped only by being produced and used in a relatively restricted territory and by not being advertised on a comprehensive national scale.

It is the newcomer in slag territory that is confused. To him it is a new material and something to be approached with caution and surrounded by many specification limitations and in general closely watched.

Some fifteen years ago my attention was focused on slag as an aggregate in concrete when I assumed charge of a test program

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that extended over ten years. In that interval, and during the last few years, I have become well acquainted with the material. My familiarity with slag, inaugurated in the testing laboratory, has been enhanced by first hand contact with it in the field as a material which we are using in important concrete and reinforced concrete construction and also by close observation of such concrete on many inspection trips taken for various committees in this organization and other technical societies.

In the published presentation of the work of Committee 201 there has been drawn a brief but comprehensive picture of the processing, testing and uses of slag and one which has not been equalled for the more commonly used aggregates for concrete. It is particularly pleasing to note that there is so little of personal belief and opinion expressed in the report. Supporting data sustaining a comprehensive story of the subject assigned to this Committee makes the report a most valuable contribution to the technical literature of concrete construction.

A particular hobby of mine for many years has been to investigate the durability of concrete in every type of structure and in all classes of construction. In this effort to learn as much as possible about conditions affecting the durability of concrete I have seen innumerable structures in which slag was used as the coarse aggregate—probably several hundreds of them in all. Among these have been bridges, factories, warehouses, highways and possibly every example of concrete one can conceive of.

Although I have found slag structures that were not in strictly first class condition—some few of them pretty bad in fact —there has never been brought to my attention an example of troubled concrete that could be attributed directly to failure or disintegration of the slag as an aggregate. Such examples may exist, and if so, I would appreciate having them called to my attention in order that studies may be made in connection with the work of Committee 801, on Durability of Concrete.

Tests conducted under my supervision have shown that specimens of reinforcing steel embedded in samples of slag concrete for five years showed no indication of corrosion, except slight rust spots where air-pockets had formed next to the steel. This condition was common to all aggregates, gravel, traprock, granite, limestone and dolomite.

For many years I have claimed that durability should be the predominating factor in concrete construction and that the selection of both fine and coarse aggregates should be based on their ability to resist the forces tending to break down or disrupt the mass.

There has been brought to my attention a digest of something over four hundred specifications covering the uses of slag. Painstaking effort has been put forth to assemble in tabular form all of the words used in these requirements. In reading these data I was greatly interested to note that of all the words used the one which occurred most frequently was "durable." A total of 267 engineers, or groups of engineers, had imposed this requirement in their specifications.

I have seen a lot of rather severe limitations originally imposed on the material dissipated in the face of developed facts and data supported by service history until now the tendency is towards a specification in name only.

It is not my thought to give blanket approval to this material by name alone. However, when slag is adequately prepared, correctly processed, kept uncontaminated and suitably sized, it makes an excellent aggregate. If any criticism is to be made of the report of Committee 201 it is that the members who presented it did not make definite recommendations for specifications. Perhaps there were restrictions imposed on them and they might have exceeded the authority vested in the Committee, and I feel that the members should be congratulated on the scope and comprehensiveness of their work.

Discussion of Paper by Arthur Ruettgers and A. A. Whitmore

"Construction of Main Canal Lining on Kittitas Division, Yakima Reclamation Project, Washington"*

BY S. TREVOR DIBBLE†

Although situated in New Zealand at a remote distance from the main activities of the A. C. I. it is with very keen interest that descriptions of your large undertakings are followed by members here.

As that interest grows and such an excellent medium for the exchange of views is afforded by the Journal of the Institute, every member who contributes a paper for discussion by the Institute earns the thanks and gratitude of a large circle of readers who remain inarticulate through diffidence to enter the discussion for the purpose of obtaining information and replies to questions which may not be of general importance.

I wish, then, to risk that reproach and congratulate the authors, Messrs. Arthur Ruettgers and A. A. Whitmore on behalf of the members of this outpost of the Institute, for an excellent description of the efforts and methods to control the construction of this work under discussion, and also for the splendidly selected photographs which illustrate and amplify the article so well.

The authors referred to considerable transverse cracking of the lining, and found that that increased in the sections which were coated with bitumen as a curing method. May I ask whether the large amount of steel in so thin a lining, combined with the richness of the mix, would contribute largely to that defect?

*A. C. I. Journal, October. 1930, *Proceedings*, Vol. 27, p. 117.
†Registered Surveyor, Karori, Wellington, New Zealand.

It seems to me, notwithstanding that several panels collapsed through hydrostatic pressure from the back, that an adequate method of porous and tile drainage could have been provided, and the reinforcement dispensed with entirely.

With regard to transverse cracks generally in thin slabs it is, in my opinion, during the early curing periods when most of the mischief is done, and although the sprinkler idea is new to me and appears to be an excellent provision for curing in this case I would suggest that beneficial results could easily be obtained by using the burlap covering as a shade over the whole canal being concreted for fourteen days after concreting and allow the sprinklers to operate as well.

The authors drew attention to the benefits of operating two separate units on either side of the canal simultaneously for lining the sides, claiming that the spirit of competition between the gangs increased production. May I ask whether any of the bonus or differential piece rate systems for payment of labor were employed, by the contractors and if so, what were they? And were there any outstanding results noticeable from it that would be worthy of the authors' comment? Further did the tenderers for this work who appear to have competed keenly, fully understand that rigid control of quality of concrete and method of production would be literally enforced, and if so did it affect the tendering?

I should also like to know whether the rigid enforcement of quality control could always be safely entrusted to the usual job inspector, or had these men to be specially selected for their ability to accommodate their views to the new horizons in concrete work?

I may say I have found it difficult to obtain inspectors who have the necessary practical experience and knowledge of men, together with an aptitude for the learning and application of the new methods of quality control and that many petty deceptions are practiced which are hard to detect and result in false reports.

No doubt these are growing pains we feel here which your older practice of quality control will have outgrown, and I trust the authors may be able to give me an answer which will serve to guide my feet from pitfalls in the path.

AUTHORS' CLOSURE

The comments of Mr. Dibble are gratefully acknowledged by the authors, and are especially appreciated as they radiate a common interest from so distant a center of engineering activity as New Zealand.

In commenting on the transverse cracking between panel joints of the 3-in. concrete lining, Mr. Dibble asks "whether the large amount of steel in so thin a lining, combined with the richness of the mix, would contribute largely to that defect."

The authors have made a further study of the transverse cracking since their original paper was prepared and have arrived at the conclusion that the extent of such cracking can be attributed primarily to the action of the longitudinal reinforcing steel and friction of the subgrade in resisting the contraction of the concrete under falling temperatures and gradual air drying, rather than to any material fault in the fabrication, placing, or curing methods employed. One argument in favor of this conclusion is that the cracks developed progressively. On the south or shaded side slope of the lining, from 20 to 30 per cent of the cracks made their appearance during the first three weeks and the remainder during the following three weeks, while on the north or sunny slope about 60 per cent of the cracks developed during the first 3 weeks' period. Furthermore, lining placed in September was found to have approximately the same number of transverse cracks after six weeks had elapsed as lining placed in the middle of the summer when the maximum atmospheric temperatures were much higher but the daily range was nearly the same. The only evidence of crazing, indicating rapid surface drying of the green concrete, was found in several panels of Hunt-processed lining placed on July 25 when the temperature mounted to 107 deg. F. It does not appear that the richness of the mix used (if a 1:2.4:3.6 dry rodded mix may be classified as rich) has any particular bearing on the development of the transverse cracks if it be assumed, as seems reasonable, that the tensile strength of the concrete and the bond between concrete and reinforcing steel develop proportionally. If the friction of the subgrade, being a relatively minor factor, is disregarded and it is assumed that the average unit bond strength is equal to the unit

tensile strength of the concrete, it may be argued that the ultimate spacing of transverse cracks will be twice the length of steel required to develop a bond equal to the tensile strength of the concrete, or

$$L = 2\left(\frac{A}{\pi d}\right) = 2\left(\frac{\pi d^2}{4p} \times \frac{1}{\pi d}\right)$$

$$\text{or } L = \frac{d}{2p} \qquad (1)$$

in which L = ultimate spacing of cracks, A = cross-sectional area of concrete, d = diameter (or width) of bar, and p = percentage of steel.

For $d = \frac{3}{8}$ in. and $p = .31$ per cent, the values applying to the lining, we have $L = .375/.0062 = 60$ in. or 5 ft. This spacing is in reasonable accord with the average 8-ft. and 6-ft. crack spacings found in the water-cured and Hunt-processed linings respectively, especially when it is considered that additional cracks probably developed subsequent to the time the survey was made, due to increased drying shrinkage and temperature drop.

Inasmuch as equation (1) is obviously predicated on the reinforcing steel remaining intact, it becomes necessary, in investigating the possibility of restricting the cracks to the construction joints at 14 ft. intervals, to consider the question of stresses in the steel.

Assume that the steel takes its own temperature stress in addition to the stress transferred by the concrete after cracking. Then

$$f_s = f_c\,(n + 1/P)$$

in which f_s = unit tensile stress in steel, f_c = unit tensile stress in concrete, n = ratio of moduli of elasticity of steel to concrete, and P = percentage of steel.

Assume further that $n = 10$ and that the unit tensile stress in the steel, of hard grade, should not exceed 100,000 lbs. per sq. in. when the concrete has acquired a unit tensile strength of say 300 lbs. per sq. in. Then

$$100{,}000 = 300\left(10 + \frac{1}{P}\right)$$

$$\textit{and } P = .0031$$

Substituting this value of P in equation (1) we have

$$L = \frac{d}{2\ (.0031)} = 162\ d.$$

If $d = \frac{1}{4}$ in. ($5\frac{1}{4}$ in. spacing), $L = 41$ in. $= 3$ ft.-5 in.
If $d = \frac{3}{8}$ in. (12 in. spacing), $L = 61$ in. $= 5$ ft.-1 in.
If $d = \frac{1}{2}$ in. ($21\frac{1}{4}$ in. spacing), $L = 81$ in. $= 6$ ft.-9 in.

It appears from the foregoing that it is impracticable to design a 3-in. reinforced concrete lining, under the conditions mentioned, to insure an ultimate uniform distribution of transverse cracks at 14 ft. intervals. If steel in the amount of .31 per cent is used, intermediate cracks must be anticipated or the panel length reduced to not over 7 ft. Or if the percentage of steel is reduced to conform to a construction joint spacing greater than 7 ft., ultimate failure of the steel and non-uniform distribution and width of joint openings must be expected.

Mr. Dibble expressed the opinion "that an adequate method of porous and tile drainage could have been provided, and the reinforcement dispensed with entirely." The nature of the country traversed and the variation in soil conditions made the problem of draining the lining one of the most difficult ever encountered by the Bureau of Reclamation and the matter was given very careful attention. With a few minor exceptions the drainage facilities provided appear to have proven adequate and successful to a degree surpassing all expectations.

While the use of plain concrete would undoubtedly have eliminated the intermediate transverse cracks, or at least materially reduced their number, the authors do not look upon omission of the reinforcement with favor, particularly in a region subject to severe frost and unfavorable drainage conditions. The intermediate cracks are not regarded as a serious menace to the integrity of the lining and it is believed that the additional expense for reinforcement (about 10 per cent of the cost of the lining) will prove a judicious investment in providing continuity and increased strength as security against heaving, uneven settlement, undermining, earth or water pressure, flotation, impact from falling rocks, serious washouts, or other conditions which might develop.

With regard to the spirit of competition displayed by the two lining-machine operating crews working simultaneously on both

side slopes of the canal, it may be said that the attitude manifested was apparently spontaneous and not induced by any piecework or bonus system of payment, so far as the authors are aware. The rate of production was held within bounds by the inspector's insistence on proper placement and compacting of the concrete, as well as by the desire of the foreman to avoid reworking and extra finishing.

Although very little trouble was experienced with concrete inspectors, it is the opinion of the authors that rigid enforcement of quality of concrete cannot be safely entrusted to the usual job inspector without apprizing him in all seriousness of the requirements, providing him with the needed implements and information in an understandable and convenient workable form, supplying supervision commensurate with his reactions, and above all instilling an interest and pride in his work. The inspectors were selected from a civil service register of applicants meeting the minimum educational and experience requirements. There were, of course, differences in type and character of the men, but on the whole the inspectors employed proved to be conscientious, trustworthy and efficient. A good many, in fact, developed into enthusiastic control advocates. What few deceptions were practiced were soon detected and obliterated, if necessary by peremptory discharge. It goes almost without saying that wholehearted support and positive backing by the engineer in charge is essential to the complete success of any attempt at rigid concrete control.

In closing, the authors desire to call attention to one feature of the lining not mentioned in Mr. Dibble's discussion but which may be of general interest, because of its practical significance. The original paper stated, in connection with the final adoption of the plain butt type of panel joint, that the use of the edging tool for grooving the top surface of the concrete at the joint was early discontinued upon finding that the groove was not required to insure the development of the crack. Information just received, however, is to the effect that some appreciable spalling has occurred along sharp edges of the cracks and that the edging tool, which provides rounded shoulders, is again being utilized on lining work.

Reinforced Concrete Column Investigation*

Second Progress Report of Committee 105

Committee 105 presented its first progress report at the 1930 convention of the American Concrete Institute in New Orleans. This report was published in the April, 1930 Journal, and is found on pages 601 to 615 of the 1930 *Proceedings*. It contained the program of tests as laid out by the committee, and the results of some of the preliminary investigations on the materials to be used. Since that time, three committee meetings have been held and the investigational work is well under way. In carrying out the tests slight modifications of the program have been found necessary, but in the main the original program has been followed.

At the University of Illinois all the specimens for this program have been made. At Lehigh University all the specimens except the greater portion of those in Series 4 have been made. At both universities many of the specimens have been tested and the work of interpreting the data is under way.

At a meeting of the committee held in December, 1930, at the Bureau of Standards, Washington, D. C., it was decided unanimously, that these participating laboratories, the University of Illinois, and Lehigh University, should be authorized to submit their own progress reports of the tests. Since the tests for Series 1 and 2 had already been completed, it was found advisable to publish the results for these two series in the February Journal of the Institute. These laboratory reports follow the report of the committee. They have been prepared independently by each laboratory, and are intended to present as concisely as possible, the results of the tests to date for future use of the committee and the members of the Institute. These reports do not bind the committee to the expression of any opinion as to their significance.

*For presentation and discussion at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.

The work of Series 3 has also been carried far enough to warrant a similar progress report, but it was believed desirable to delay this report slightly in order to include as late data as possible. In conformity with this program it is expected that a further progress report covering Series 3 will be available in preprint at the Milwaukee convention in February.

COMMITTEE 105—REINFORCED CONCRETE COLUMN INVESTIGATION

W. A. SLATER, *Chairman*
P. H. BATES
R. L. BERTIN
F. E. RICHART
R. D. SNODGRASS
W. S. THOMSON
F. R. MCMILLAN, *ex-officio*
as Chairman Dept. 100, Research

Readers are referred to the JOURNAL *for March, 1931, for a further progress report, and to the* JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

First Progress Report on Column Tests at Lehigh University

BY W. A. SLATER* AND INGE LYSE†

INTRODUCTION

The tentative program and the results of the preliminary tests of the concrete for the reinforced concrete column investigation being carried out under the sponsorship of the American Concrete Institute was published in the Journal for April, 1930[1]. The parts of the report presented herein have been prepared by those having charge of the tests at Lehigh University. The authors accept the responsibility for the statements made. An effort has been made to avoid all generalizations but to present the data as fully as possible at the present time and to point out the important relations indicated in Series 1 and 2.

SERIES 1. STUDY OF END CONDITIONS

1. *Purpose and Program*—The purpose of Series 1 of this investigation was to study the effect of different end conditions on the strength and other properties of the columns, and from the results obtained decide upon the end condition to be used in the remaining series. This series was therefore completed before the others were begun.

The columns included in this series had a diameter of 8 in., an overall length, or shaft length, of 60 in., and were tested at an age of 28 days. All columns contained eight ½-in. square longitudinal reinforcing bars and a $\frac{3}{16}$-in. wire spiral of 1.35-in. pitch. This corresponds to about 4 per cent of longitudinal, and 1.2 per cent of lateral reinforcement. The concrete was designed for a strength of 3500 lb. per sq. in. at 28 days as determined on 6 by 12-in. cylinders at the time of testing the columns.

*Director, Fritz Engineering Laboratory, Lehigh University. Chairman, Committee 105.
†Assistant Engineer, Portland Cement Association, In immediate charge of column tests at Lehigh University.
[1]*A. C. I. Proc.*, Vol. 26, p. 601.

Series 1 included three types of end condition. In the first type there were no capitals, and the ends of the longitudinal reinforcing bars were flush with the ends of the column. The second type had 14-in. diameter capitals. The third type had dowels and no capitals. Two groups of columns of the third type were tested. One group had 13-in. dowels which lapped over the reinforcing bars for a length of 20 diameters. The other group had 18-in. dowels providing for a 30-diameter lap. There was the same number of dowels as of the longitudinal bars, and both were of the same grade of steel. Two columns were included in each group.

2. *The Concrete*—The concrete used in this series was a 1:2.4:3.6 mix by weight. The cement content was 5.4 sacks per cu. yd. of concrete. The nominal water content was 40.5 gal. per cu. yd. of concrete or 7.5 gal per sack of cement. The concrete was mixed in a kettle-shaped mixer of about 2.5 cu. ft. capacity. The dry materials were mixed for one minute. Water was then added and the mixing was continued for three minutes more. Six 6 by 12-in. cylinders were made with each column, three for test at the age of 28 days, and three for test at 56 days.

Table 1 gives information on the properties of the concrete used in this series.

3. *The Reinforcement*—The longitudinal reinforcing bars were donated by the Bethlehem Steel Company of Bethlehem, Pennsylvania. These were plain bars having an average yield-point stress of 48,000 lb. per sq. in. and an average ultimate strength of 76,850 lb. per sq. in. The test results of five coupons taken from five different bars show a good degree of uniformity.

Coupon Number	1	2	3	4	5	Av.
Yield-Point Stress*	47,520	48,800	48,040	47,560	48,120	48,000
Ultimate Stress*	76,680	77,000	76,760	76,800	77,000	76,850

*Stresses given in pounds per square inch.

The lateral reinforcement was donated by the American Steel and Wire Company of Chicago, Illinois, and fabricated into spirals by the American System of Reinforcing. It consisted of $\frac{3}{16}$-in. wire spirals having a pitch of 1.35 in. A definite yield point could not be detected for the spiral reinforcement. Therefore only ultimate strength was obtained.

Coupon Number	1	2	3	4	5	6	7	8	Av.
Strength lb. lb. per sq. No in.	Test	89,340	83,570	85,730	83,930	83,570	87,170	77,450	84,390

It will be noted that the lateral reinforcement was less uniform but about 10 per cent higher in average strength than the longitudinal reinforcement.

4. *Fabrication of Reinforcement*—All the reinforcement for each column was assembled into a unit before being placed in the mold. Four of the longitudinal bars had six ½-in. steel cubes welded to the side so as to form a continuous row of five 10-in. gage lengths. The end cubes were placed five inches from the ends of the column or ends of the shaft.

The spirals had three spacers which were placed approximately 120 degrees apart. Eight longitudinal bars were fastened with wires to the inside periphery of the spirals approximately 45 degrees apart.

5. *Making Columns*—In Group 1 the longitudinal bars were flush with the ends of the column. This provided for transferring the load from the machine directly upon both the concrete and the steel. In making all columns, the concrete was filled to the top of the molds. A settlement of about $\frac{1}{16}$-in. took place during the setting, due to the consolidation of the mass. Neat cement paste having a water-cement ratio of 0.3 by weight, re-mixed

TABLE 1—PROPERTIES OF CONCRETE

Group No.	End Condition	Average Slump In.	28-Day Strength of Control Cylinders lb. per sq. in.								Average for Group
			Cylinders for Column A				Cylinders for Column B				
			1	2	3	Av.	1	2	3	Av.	
1	Plain	4¾	3075	2920	2770	2920	3510	3340	3200	3350	3140
2	Capital	4¼	3220	3030	3260	3170	3380	3740	3140	3420	3300
3	(20-dia. splice)	4½	3460	3560	3500	3510	3460	3620	3440	3510	3510
4	(30-dia. splice)	3½	3260	*	2980	3120	3340	3410	3280	3340	3230

*Broken Cap



FIG. 1—MAKING COLUMNS HAVING CAPITALS



FIG. 2—MAKING COLUMNS HAVING DOWELS

at four hours, was used as capping material. Planed steel plates used for the capping were worked down upon the ends of the longitudinal bars, bringing them flush with the surface of the concrete. The caps were usually about $\frac{1}{16}$-in. thick.

The columns of Group 2 had a capital at each end. The shaft had the dimensions of the plain columns. The capitals were 18 in. long and 14 in. in diameter. The longitudinal bars extended a distance of 30 diameters, or 15 in. into the capital. The spiral extended six inches into the capital. Wire hoops were placed in each capital to prevent its falling apart in case of splitting during the testing. The form used for making the columns of Group 2 is shown in Fig. 1.

The columns of Group 3 had dowels of such a length as to form a 20-diameter splice. The longitudinal reinforcing bars extended to within three inches of each end, and the dowels were flush with the ends. Group 4 was of the same type as Group 3 except

that a 30-diameter splice was used. Screws which passed through the end plates and into the ends of the dowels, held the dowels flush with the surface of the concrete and kept them in position during the placing. Fig. 2 shows the placing of the top plate on a column having dowels. The mold was filled to the top and the dowels were then forced into the concrete until the plate rested on the top of the mold. At the time of capping, the screws were removed and the end plate lifted. The capping was done in the ordinary manner.

In making the columns, the concrete was placed continuously and was rodded into place with ½-in. steel rods. The mold was tapped with a hammer in order to consolidate the mass. At the age of 24 hours the molds were removed, and a ¼-in. diameter by ½-in. steel plug was set in the concrete with neat cement paste beside each lug on the longitudinal bar. The columns were then given three coats of linseed oil and placed in the moist room. One day before testing, each column was removed from the moist room and gage holes drilled in all the plugs and lugs. Five continuous 10-in. gage lengths were thus obtained on both the steel and the concrete on elements 90 degrees apart, around the column. Elements 1 and 3 were at opposite ends of one diameter, and 2 and 4 at opposite ends of the other.

6. *Testing*—The columns were cured in the moist room until they reached an age of 28 days, and then were tested to failure in an 800,000-lb. vertical screw testing machine. For the columns without capitals, an eight-inch spherical bearing block was placed between the head of the machine and the top of the column. The columns with capitals required a larger spherical bearing block and a 17-in. block was used. It was so heavy that it was placed at the bottom instead of the top. The column was plumbed by means of plumb bobs.

Before each column with plain ends was loaded, a complete set of strain-gage readings was taken. For the columns with capitals, a load of 200 lb. was applied before the initial readings were taken. With the spherical bearing block at the base of the column, this load was necessary to hold the column in line.



FIG. 3—COLUMN A OF GROUP 1 AFTER TEST

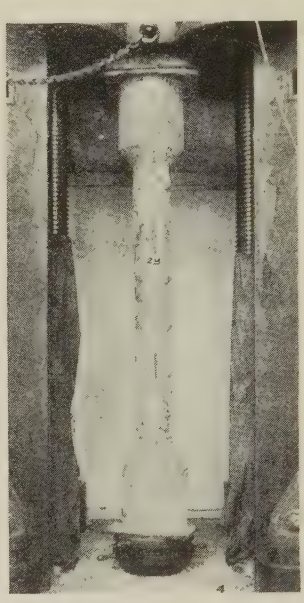


FIG. 4—COLUMN B OF GROUP 2 AFTER TEST

Five 10-in. gage lengths were observed on four rows of lugs on the steel bars with corresponding readings on the plugs in the concrete. This makes a total of 20 gage lengths on the steel, and 20 in the concrete. Each gage length was observed twice, so that 80 observations were made for each set of readings.

The testing crew consisted of two observers and a recorder. The observers took readings on opposite ends of the same diameter simultaneously. The order used in taking the measurements was as follows: Readings were taken from top to bottom on the steel in rows 1 and 3, and then from bottom to top on the concrete. The same order was used for readings on rows 2 and 4. A second set of readings was taken in a similar manner. Ordinarily the maximum variation between the readings of the two sets was 0.0002 in. as read on the dial of the Whittemore strain gage used. If a larger variation occurred a check reading was taken.

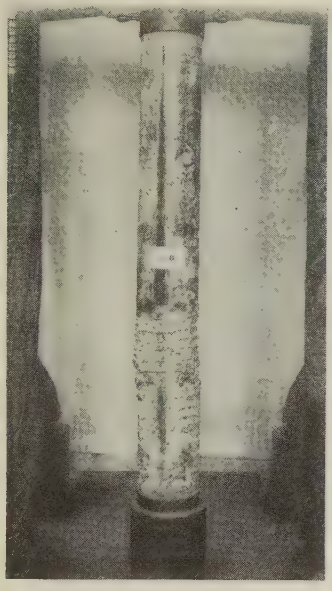


FIG. 5—COLUMN B OF GROUP 3 AFTER TEST

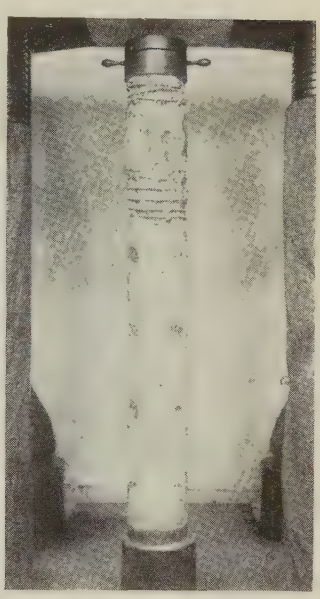


FIG. 6—COLUMN A OF GROUP 4 AFTER TEST

The load was applied in increments of approximately 25,000 lb. A complete set of observations was made for each load. The beam of the machine was rebalanced after the observations so that the load, both at beginning and end of observations, was recorded. This procedure was followed until the deformations went beyond the range of the strain gage on some of the gage lengths.

The time required for each loading, that is, from one application of the load to the next, varied. After some experience, the time for each set of observations ranged from seven to eight minutes.

7. *Strength of Columns*—All columns had 4 per cent of longitudinal reinforcement and 1.2 per cent of spiral reinforcement. The concrete was of the same mix and water-cement ratio throughout the series. Due to minor variations, however, slight differences in strengths were obtained. The test results of the control cylinders are given in section 2. The major difference in



FIG. 7—DETAILS OF FAILURE OF COLUMNS HAVING DOWELS

strength of the columns is, therefore, attributed to the end condition. Table 2 gives the maximum strengths of the columns having different end conditions. Figs. 3 to 7 inclusive, show the types of failure for the groups. The columns of Group 1, shown in Fig. 3, failed near the center. The columns with capitals, shown in Fig. 4, failed about ten inches below the capital. The columns with dowels failed at the ends of the dowels, either at the top or bottom of the column, as illustrated in Figs. 5 and 6. Fig. 7 shows the details of the failure of columns having dowels. After removal of the load a gap was generally found between the end of the reinforcing bar or dowel and the concrete. This is illustrated in Fig. 7. At the ends of the columns also the bars had been pulled back into the concrete slightly. This was true in the columns with continuous bars as well as in those with dowels. It appears that for some reason the concrete elongated more than did the bars on removal of the load. The cause of this phenomenon has not been explained, but it may have been

TABLE 2—STRENGTH OF COLUMNS AND THEIR CONTROL CYLINDERS

Group No.	Maximum Load, lb.			Strength of Control Cylinders lb. per sq. in.		
	Col. A	Col. B	Av.	Col. A	Col. B	Av.
1	252,800	265,000	258,900	2,920	3,350	3,140
2	260,000	251,000	255,500	3,170	3,420	3,300
3	218,600	245,400	232,000	3,510	3,510	3,510
4	231,000	231,000	231,000	3,120	3,340	3,230

due partly to the buckling of the bars as shown in Fig 3 and partly to the assistance of the spiral reinforcement in maintaining the integrity of the concrete.

In order to make a comparative study of the effect of the end condition on the strength of the columns, the variation in the strength of the concrete must be taken into account.

It is shown in Series 2 that the load carried by the plain columns was very nearly 85 per cent of the strength of the control cylinders regardless of the strength of the concrete. Assuming that the concrete in the core of a spirally reinforced column will carry the same amount of load as in a plain column, the remaining load may be attributed to the presence of the steel, both spiral and longitudinal. In computing the load carried by the concrete the area used in the core area specified by the American Concrete Institute Building Code, that is, "the area within the outer circumference of the spiral hooping." As the outer diameter of the spirals was 7.75 in. and as the sectional area of the longitudinal bars used was 2 sq. in., the net core area of the concrete was:

$$\frac{\pi}{4}(7.75)^2 - 2 = 45.1 \text{ sq. in.}$$

The load attributed to the concrete was then 0.85 x 45.1 f'_c, where f'_c = unit strength of the control cylinders. Table 3 gives the computed division of load between concrete and reinforcement at the time of failure. The values given in this table have been plotted in Fig. 8. The upper line in this figure represents the proportionate total loads, and the lower line represents the proportionate loads attributed to the reinforcement in the different groups. The fact that the curve for load carried by the steel

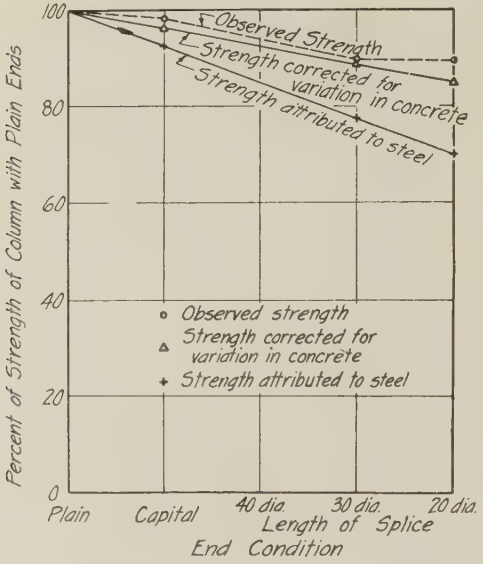


FIG. 8—RELATION BETWEEN END CONDITION OF COLUMN AND STRENGTH

is a straight line is due to a suitable selection of horizontal scale. Fig. 8 indicates that in order to obtain the same effect due to the reinforcement in a column with spliced ends, as in a column with bars flush with the ends, the length of the splice would have to be 60 times the diameter of the bars. The capitals apparently

TABLE 3—DIVISION OF LOAD BETWEEN CONCRETE AND REINFORCEMENT

Group No.	End Condition	Maximum Load		Load Attributed to Concrete	Load Attributed to Steel	
		lb.	per cent	lb.	lb.	per cent
1	Plain	258,900	100	120,500	138,400	100
2	Capital	255,500	98.5	126,800	128,700	93.0
3	(20-dia. splice)	232,000	89.5	135,000	97,000	70.0
4	(30-dia. splice)	231,000	89.2	124,000	107,000	77.4

TABLE 4—COLUMN STRENGTH CORRECTED FOR VARIATION IN CONCRETE STRENGTH

Group No.	Load Attributed to		Total Load	
	Concrete lb.	Steel lb.	lb.	per cent
1	134,500	138,400	272,900	100
2	134,500	128,700	262,200	96.1
3	134,500	97,000	231,500	84.8
4	134,500	107,000	241,500	88.5

gave the equivalent of a splice of 50 diameters.

The load carried by the steel would probably have been no different if the cylinder strength had been exactly 3500 lb. per sq. in. for all groups. Assuming that this is true the strength of the reinforced column for 3500-lb. concrete has been estimated by adding to the load carried by the reinforcement 85 per cent of the assumed cylinder strength of 3500 lb. per sq. in. The computations are shown in Table 4 and the column strengths corrected in this manner are shown in Fig. 8.

Fig. 8 indicates that the columns with the longitudinal steel flush with the ends of the column, developed higher strength than the columns with capitals or spliced reinforcement. The comparative ease of making this type of column makes it less expensive than the columns having dowels and much less expensive than the columns with capitals.

8.—*Deformations*—In order to study the relation between deformations at different elevations, the average deformations for each group of columns have been plotted in Figs. 9 to 12 inclusive.

Fig. 9 shows the average deformations for the columns of Group 1. This figure illustrates that at low loads the uniformity in the elastic properties at different elevations was good. For loads of 100,000 lb. or less, the deformations were practically equal at all elevations of the column. For higher loads, however, the deformations at the center elevation of the column were considerably larger than those at the ends. The most satisfactory explanation of this phenomenon seems to be the possibility of lack of homogeneity of the concrete. At low loads the steel

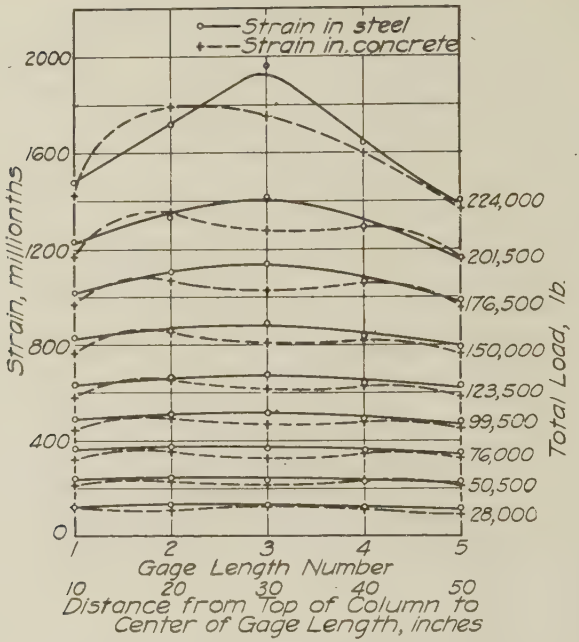


FIG. 9—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 1

and the concrete deformed together. At higher loads this equality of deformations continued near the ends, but near the center the deformations in the steel exceeded those in the concrete. The excess of deformation in the steel over that in the concrete increased as the load increased and failure occurred at the center as shown in Fig. 3.

Fig. 10 shows the average deformations of the columns having capitals. It is seen from the figure that the deformation of the steel was nearly equal to that of the concrete for all loads. It is noted, also, that the deformations at different elevations were nearly equal at low load, while for high loads the deformation was much greater at the top than at the bottom.

Fig. 4 shows that the failure occurred near the top of the shaft as would be expected from the behavior described. Slightly

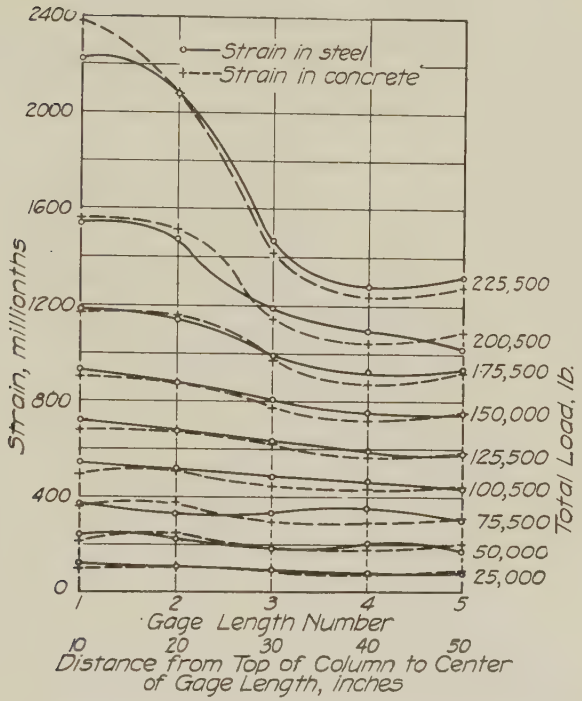


FIG. 10—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 2

weaker concrete just below the capital, may have caused the deformations near the top to be larger than those near the bottom. A noticeable eccentricity in the loading (see Fig. 18) probably added to the stresses at the top of the shaft.

Fig. 11 shows the deformations at different elevations for the columns of Group 3. These columns had dowels which provided for a 20-diameter splice. As before, the deformations were nearly equal at different elevations for low loads. In general, the steel and concrete deformed together until a load of about 50 per cent of the maximum strength of the column was reached. At this load the concrete in the bottom section deformed much more than did the steel, indicating that slipping of the bars must have taken place. As the load increased, the slipping between steel and

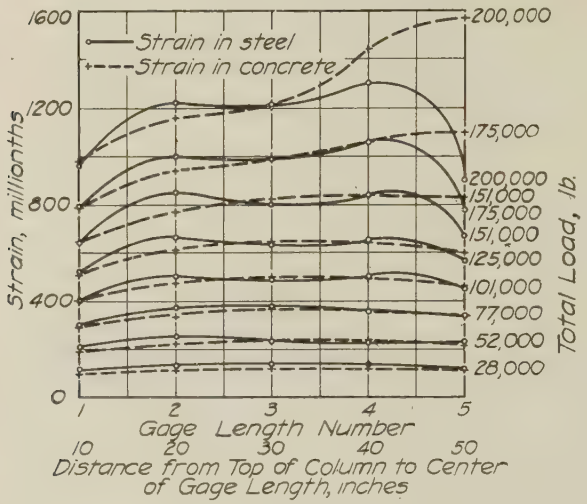


FIG. 11—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 3

concrete became more pronounced, and near maximum load it was noted that the concrete had moved away from the lugs on the steel forming a perceptible gap. The concrete spalled at the ends of the dowels and at the ends of the longitudinal bars at practically the same load, indicating that dowels and bars began to slip simultaneously.

Fig. 12 shows the deformations at different elevations for the columns of Group 4. These columns had dowels which lapped over the longitudinal reinforcing bars for a distance of 30 diameters. The figure indicates that the concrete and the steel deformed together throughout the test, for points at the center and below, and also for points above the center, until about 75 per cent of the maximum load had been reached. At this load a slip occurred between steel and concrete near the top of the column, and the deformations of the concrete there became much larger than those of the steel. The concrete deformed very fast while the stress in the steel did not increase materially. The spalling of the columns occurred at the ends of the longitudinal bars and at the ends of the dowels as in Group 3. The failure

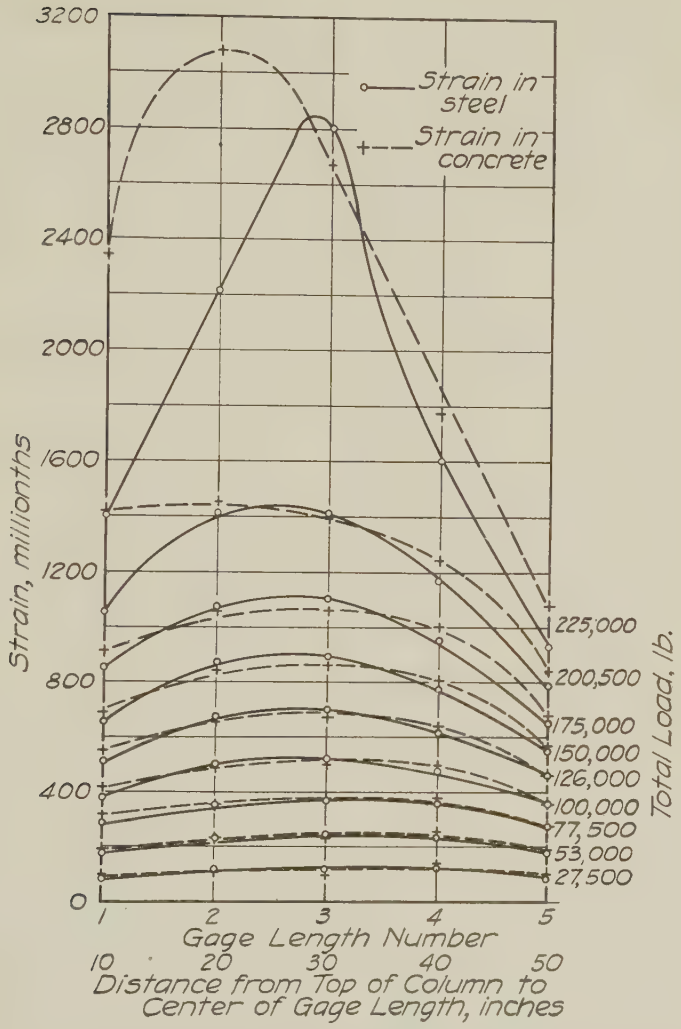


FIG. 12—DEFORMATION AT DIFFERENT ELEVATIONS FOR COLUMNS OF GROUP 4

of these columns was very similar to that of Group 3, except that the columns of Group 4 failed at the top while those of Group 3 failed at the bottom.

For both groups of columns having dowels, the deformations at all loads were smaller near the ends than at the center. This probably was due to the fact that on account of the lapping of the dowels the percentage of steel at the end section was twice the percentage at the center section.

9. *Load Distribution*—In Figs. 13 to 16, inclusive, the average deformations are plotted for the different loads on the columns. In addition, the loads corresponding to the strains in the steel are represented by a broken straight line. This line is carried to a strain of 1600 millionths, corresponding to a yield-point stress of 48,000 lb. per sq. in. within which the modulus of elasticity of the steel was assumed as 30,000,000 lb. per sq. in. No determination of the modulus of elasticity of steel was made in this series of tests. The assumed division of the maximum load carried by the columns is shown on the right-hand side of the diagrams.

Fig. 13 shows the deformation diagram for the columns of Group 1. The assumption has been made that the concrete, the longitudinal steel, and the spiral have jointly contributed to the column strength. The load attributed to the concrete at the maximum load is the product of 85 per cent of the average cylinder strength and the core area of the column, that is, 0.85 x 3140 x 45.1, or 120,500 lb. The load attributed to the longitudinal steel is the product of the yield-point strength and the area of the steel, that is, 48,000 x 2, or 96,000 lb. The sum of the loads attributed to the concrete and to the longitudinal steel (120,500 + 96,000 or 216,500 lb.) has been subtracted from the column strength (258,900 lb.). This difference (258,900 — 216,500 = 42,400 lb.) has been attributed to the spiral.

According to the above assumption, the load attributed to one per cent of longitudinal steel is $\frac{96000,}{4}$ or 24,000 lb. The 1.2 per cent of spiral apparently contributed 42,400 lb., indicating that one per cent of spiral was as effective as 1.47 per

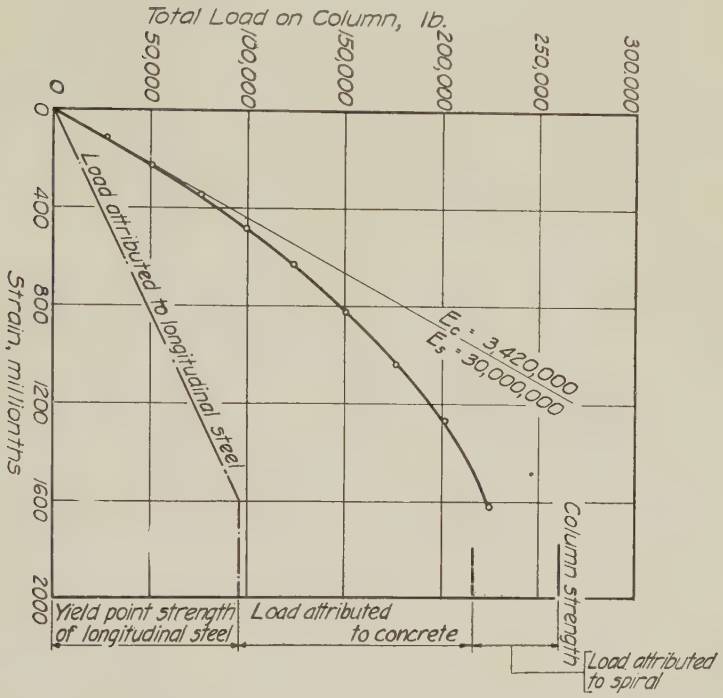


FIG. 13—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 1

cent of longitudinal reinforcement for the columns of Group 1.

The deformations at top and bottom gage lengths for the columns of Group 2 and the division of load, are shown in Fig. 14. The total load on the column has been distributed between the concrete, the longitudinal steel, and the spiral. It is noted that the load attributed to the 1.2 per cent of spiral is 32,700 lb., or 1.14 times as much as for an equal percentage of longitudinal reinforcement.

Fig. 15 shows the deformation diagram and the division of the load for the columns of Group 3. Here the sum of the yield-point strength of the steel and the load attributed to the concrete is almost equal to the maximum strength of the column. The spiral did not seem to add anything to the strength of the columns,

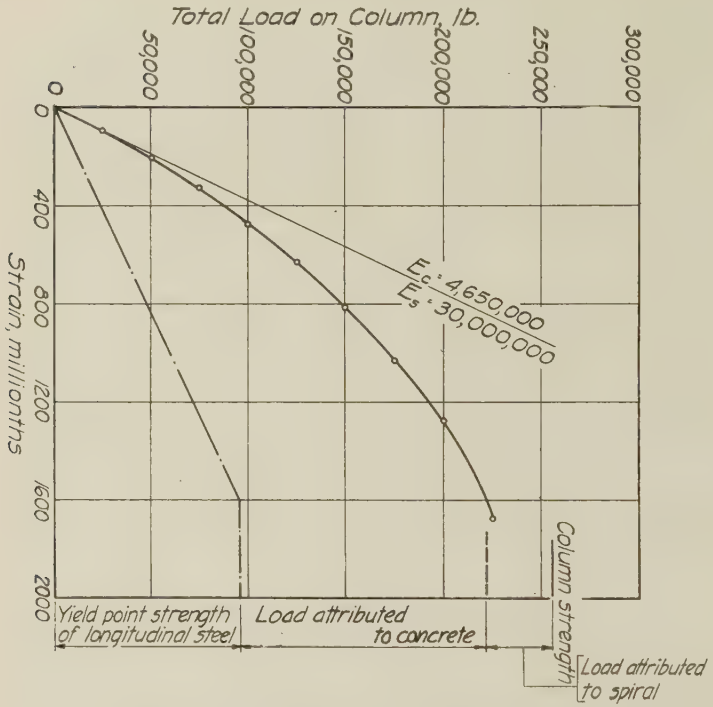


FIG. 14—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 2

probably because failure was precipitated by the slipping of the dowels before the spirals could come into action.

The deformation diagram and the division of the load for the columns of Group 4 are shown in Fig. 16. In this case the strength of the column was about 11,000 lb. greater than the sum of the yield-point strength of the steel and the load attributed to the concrete. This 11,000 lb. may be attributed to the spiral, but the spiral probably was not fully effective because of slipping of the dowels. It is of interest, however, to point out the fact that the 30-diameter lap was more effective than the 20-diameter lap.

10. *Eccentricity of Loading*—If a plane section remains plane during the loading, the sum of the strains ϵ_1 and ϵ_3 measured at the ends of one diameter, should be equal to the sum of the

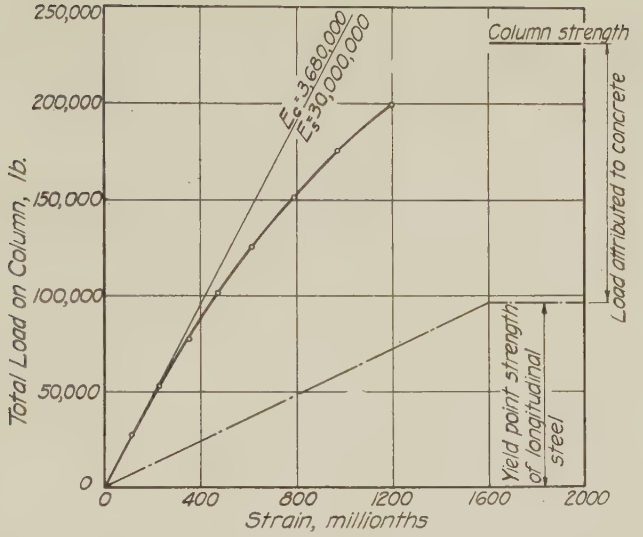


FIG. 15—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 3

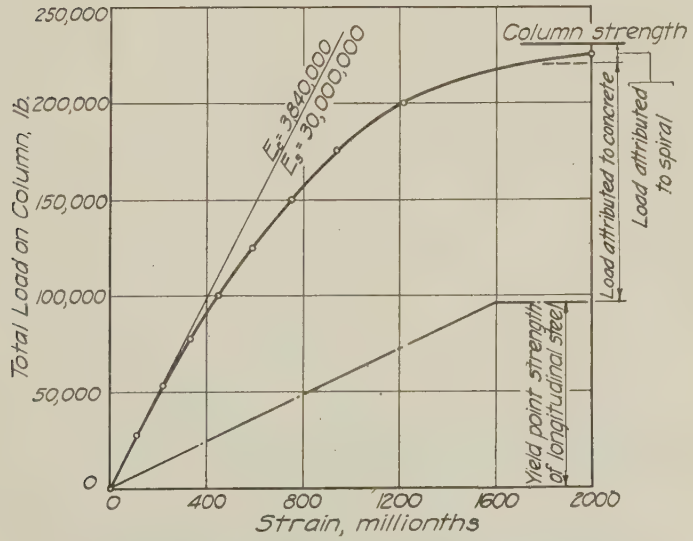


FIG. 16—DEFORMATION DIAGRAM FOR COLUMNS OF GROUP 4

strains ϵ_2 and ϵ_4 at the ends of another diameter. This relation was found to hold approximately true for all the columns. The strain volume is, therefore, a cylinder with ends parallel in the case of centric loading, and non-parallel for eccentric loading. Assuming that the stress is proportional to the strain, this relation may be used to determine the size and shape of the stress cylinder and the distance of the center of pressure from the center of the section, that is, the eccentricity of loading. The center of pressure must pass through the center of gravity of the stress cylinder in order to establish equilibrium.

The following method was used in determining the position of the center of pressure. (See Fig. 17).

The volume of the stress cylinder is:

$$V = \pi r^2 \epsilon E$$

where r = the radius of the cylinder

ϵ = the average strain, that is, $\frac{1}{4}(\epsilon_1 + \epsilon_2 + \epsilon_3 + \epsilon_4)$ and

E = modulus of elasticity

The bending strain, ϵ_b is: $\epsilon_b = r\tan\Theta$, where Θ is the angle between the end planes of the stress cylinder

The bending moment is:

$$M = \frac{\epsilon_b}{c} EI$$

where I = moment of inertia of the cross section

and c = distance from the neutral axis to the point at which ϵ_b is measured

Now

$$I = \frac{\pi r^4}{4} \qquad \text{and } c = r$$

Then

$$M = \frac{r\tan\theta}{r} \; E \frac{\pi r^4}{4} = \frac{\pi r^4}{4} E\tan\theta$$

The distance e from the neutral axis to the center of gravity of the stress cylinder, that is, the eccentricity is:

$$e = \frac{M}{V} = \frac{\frac{\pi r^4}{4} E\tan\theta}{\pi r^2 \quad \epsilon E} = \frac{1}{4} \frac{r^2}{\epsilon} \tan\theta$$

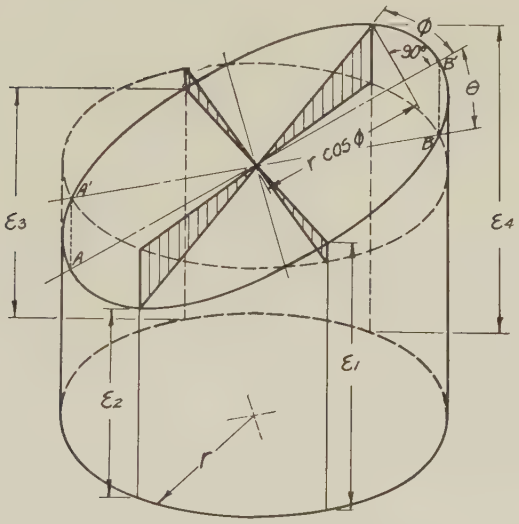


FIG. 17—ILLUSTRATION OF STRAINS USED IN DETERMINING ECCENTRICITY

From the illustration:

$$tan\theta = \frac{\epsilon_4 - \epsilon_2}{2rcos\phi} = \frac{\epsilon_1 - \epsilon_3}{2rsin\phi}$$

where ϕ is the angle between a plane passing through elements 2 and 4 and a vertical plane passing through the line of maximum and minimum stresses. The center of gravity of the stress cylinder will lie in the plane passing through the maximum and minimum strains at a distance, e, from the center line of the cylinder.

The angle ϕ is determined from:

$$\frac{\epsilon_4 - \epsilon_2}{2rcos\phi} = \frac{\epsilon_1 - \epsilon_3}{2rsin\phi}$$

$$\text{or} \quad \frac{2rsin\phi}{2rcos\phi} = \frac{\epsilon_1 - \epsilon_3}{\epsilon_4 - \epsilon_2} = tan\phi$$

$$e = \tfrac{1}{4}\frac{r^2}{\epsilon} tan\phi = \tfrac{1}{4}\frac{r^2}{\epsilon}\frac{\epsilon_4 - \epsilon_2}{2rcos\phi} = \frac{r}{8\epsilon}\frac{\epsilon_4 - \epsilon_2}{cos\phi}$$

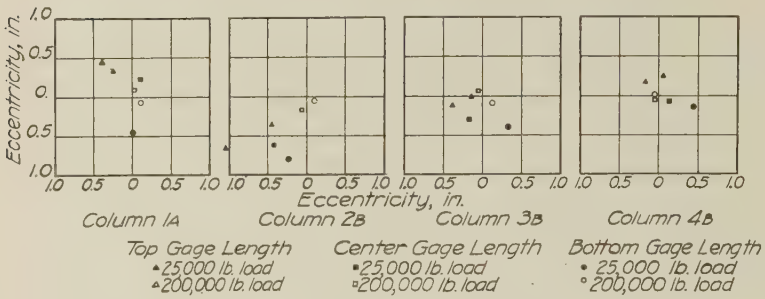


FIG. 18—ECCENTRICITY OF LOAD ON COLUMNS IN SERIES 1

Therefore the eccentricity is given by the formula:

$$tan\phi = \frac{\epsilon_1 - \epsilon_3}{\epsilon_4 - \epsilon_2} \qquad \text{and} \qquad e = \frac{r}{8\epsilon} \frac{\epsilon_4 - \epsilon_2}{cos\phi}$$

Thus the center of pressure may be easily determined from the readings obtained on the four elements 1, 2, 3, and 4. In Fig. 18 the center of pressure is given for top, center, and bottom elevations for loads of 25,000 and 200,000 lb. respectively.

This figure shows that the eccentricity was in all cases considerably less for the load of 200,000 lb. than for 25,000 lb. Also, it was generally less for the center and the bottom than for the top of the column. It may be shown that the ratio of bending stress to total stress is equal to the ratio $\frac{4e,}{r}$ that is, four times the ratio of the eccentricity to the radius of the column. For the columns, without capitals this ratio, $\frac{4e,}{r}$ was always less than $\frac{1}{2}$. For the 200,000-lb. load it was generally much less. For the columns with capitals the ratio, $\frac{4e,}{r}$ was greater than $\frac{1}{2}$ in all cases for the load of 25,000 lb. and at the top it exceeded 1, that is, the bending stress in this case was greater than the average stress, $\frac{P.}{A}$ However, for the load of 200,000 lb. the bending stress had been reduced to about one-half the average stress. For all

the other columns at the load of 200,000 lb. the bending stress was generally less than 20 per cent of the average stress $\frac{P}{A}$. The foregoing analysis has been based upon an assumed constant modulus of elasticity. The fact that in general the modulus is not constant will operate to reduce somewhat the eccentricity and the ratio of bending to average stress.

11. *Summary*—The columns having longitudinal reinforcing bars flush with the concrete at the end of the column (Group 1) appeared to develop the full strength of their component parts and failed near the center without showing local weakness.

The columns with capitals developed high bending stresses and failed near the top.

The columns with dowels showed less strength than did those with continuous bars. Failure occurred at the ends of the bars and at the ends of the dowels.

The superiority of the columns with continuous bars flush with the surface of the concrete and having no capitals, together with the fact that they were the less expensive to make and test, resulted in the adoption of this type of column for the remainder of the investigation.

SERIES 2. EFFECT OF HOLDING THE LOAD FOR FOUR HOURS

12. *Purpose and Program*—The purpose of this series of tests was to ascertain whether the holding of load for a short period such as four hours, would have any effect on the strain and the ultimate strength of the column. There was the added purpose of comparing strengths of columns with three different grades of longitudinal reinforcement, structural, intermediate, and rail steel grades. Variations in percentage of reinforcement, and strength of concrete also were introduced as is indicated in Table 5.

The columns had an outside diameter of 8¼ in., a core diameter of 8 in., and an overall length of 60 in. The longitudinal bars were 60 in. long and the ends were milled, thus insuring direct bearing of both steel and concrete on the end plates. Three like columns were made for each group of plain concrete columns and two like for each group of reinforced concrete columns.

TABLE 5—PROGRAM OF TESTS FOR SERIES 2

Concrete Strength lb. per sq. in.			Reinforcement: Longitudinal Grade	Longitudinal Per cent				Spiral Grade	Spiral Per cent		Method of Loading		No. of Like Columns	Total No. of Columns
2000	3500	5000	...	0					0		Fast	Slow	3	18
2000	3500	5000	Interm. Struct. Rail			4		Interm.		1.2	Fast	Slow	2	36
	3500		Interm.	0	1.5		6	Interm.		1.2	Fast	Slow	2	12
	8000			0					0		Fast		3	3
	8000		Interm.	0	1.5	4	6	Interm.		1.2	Fast		2	8
													Total	77

13. *Proportioning and Strength of Concrete**—The concrete strength was determined on moist-cured 6 by 12-in. cylinders at an age of 56 days. The concrete designed for a cylinder strength of 2000 lb. per sq. in. was a 1:3.42:5.13 mix by weight of the dry materials. The assumption was made that the amount of water absorbed by the dry aggregates was 0.7 per cent of their weight. The cement content was 4.0 sacks per cu. yd. and the net water content was 39 gal. per cu. yd. of concrete or 9¾ gal. per sack of cement. The test results showed an average cylinder strength of 2280 lb. per sq. in. and an average slump of 2½ in. for all specimens included in this mix.

The concrete designed for a cylinder strength of 3500 lb. per sq. in. was of a 1:2.65:4.00 mix. The cement content was 5.02 sacks per cu. yd. and the net water content 39 gal. per cu. yd. of concrete or 7¾ gal. per sack of cement. The average strength was 3680 lb. per sq. in. and the average slump 3¾ in.

The concrete designed for a cylinder strength of 5000 lb. per sq. in. was a 1:1.98:2.96 mix. The cement content was 6.5 sacks per cu. yd. and the net water content 39 gal. per cu. yd. or 6 gal. per sack of cement. The average strength was 5360 lb. per sq. in., and the slump was 5½ in. It will be noted that the water content was 39 gal. per cu. yd. of concrete for all these mixes. A

*The cement and aggregates used in this series came from the same supplies as those used in the Preliminary Investigation reported in the JOURNAL for April, 1930.

placeable concrete was produced with this water content and only a small variation was noted in the consistency as measured by the slump.

In addition, 11 columns with concrete designed to have a compressive strength of 8000 lb. per sq. in. were made. The mix used for this concrete was 1:½:1 by weight. The cement content was 15.3 sacks per cu. yd. and the water content was 46 gal. per cu. yd., or 3 gal. per sack of cement. This mix was so dry that it could not be properly placed by ordinary methods. It was therefore worked into place by means of an electric vibrator furnished by the Electric Tamper and Equipment Company. The amount of vibration required was quite extreme, and even then small pockets were found in the interior of the cylinders when they were broken. No such pockets were found in the plain columns but no examination was made of the interior of the reinforced columns.

Fig. 19 shows the relation between the strength of the concrete and the water-cement ratio used in the mix. The water-cement strength relation for the plastic mixes was a smooth curve very similar to ordinary water-cement strength curves. The relation between cement content and strength is given by the straight, broken line. In other words, as long as the water content per cu. yd. of concrete was maintained constant with nearly constant consistency of the different mixes, the strength was directly proportional to the cement content.

The strength of the concrete designed for 8000 lb. per sq. in. proved to be only 7000 lb. per sq. in., and it falls below the water-cement strength curve defined by the other mixes. Neither does this strength fall upon the straight line determined by the other mixes when their strengths are plotted against the cement content. This 7000-lb. concrete contained 15.3 sacks of cement per cu. yd. and should have a strength of approximately 12,000 lb. per sq. in. in order to fall upon either of the strength curves shown in Fig. 19.

14. *Modulus of Elasticity of Concrete*—The modulus of elasticity was determined on one control cylinder from each column. The apparatus used consisted of two steel collars, 10 in. apart vertically, attached directly to the cylinders by means

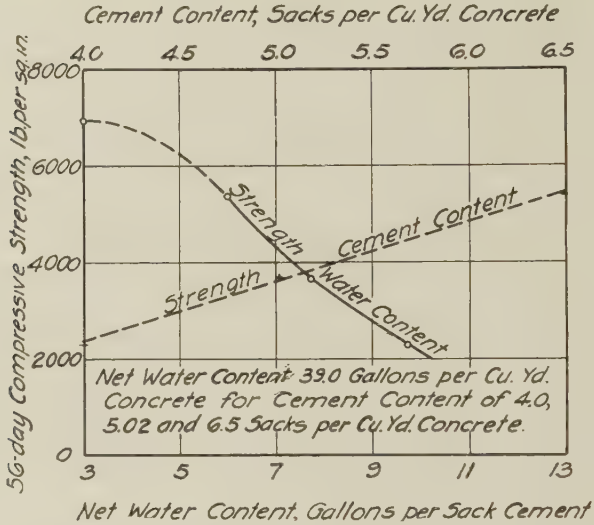


FIG. 19—RELATION BETWEEN STRENGTH AND WATER CONTENT AND BETWEEN STRENGTH AND CEMENT CONTENT FOR CONCRETE IN SERIES 2

of two set-screws 180 deg. apart. Opposite each of the set-screws a 1/10,000-in. Ames gage with its plunger pointing downward was clamped to the upper collar. A steel rod clamped to the lower collar opposite each set-screw extended upward and bore against the plunger of the Ames gage so that a direct reading of the total deformation in 10 in. was obtained. The results thus obtained were averaged for each type of concrete used and these averages are plotted in Fig. 20. The tangent modulus at a stress of 500 lb. per sq. in. was decided upon as the most reasonable value of the modulus of elasticity. In Fig. 21 the moduli of elasticity have been plotted against the strength. For all except the vibrated concrete, the increase in the modulus was very nearly proportional to the increase in strength. Even the value for the vibrated concrete falls not far above the projected straight line determined by the other values.

The results obtained indicate that the method used for deter-

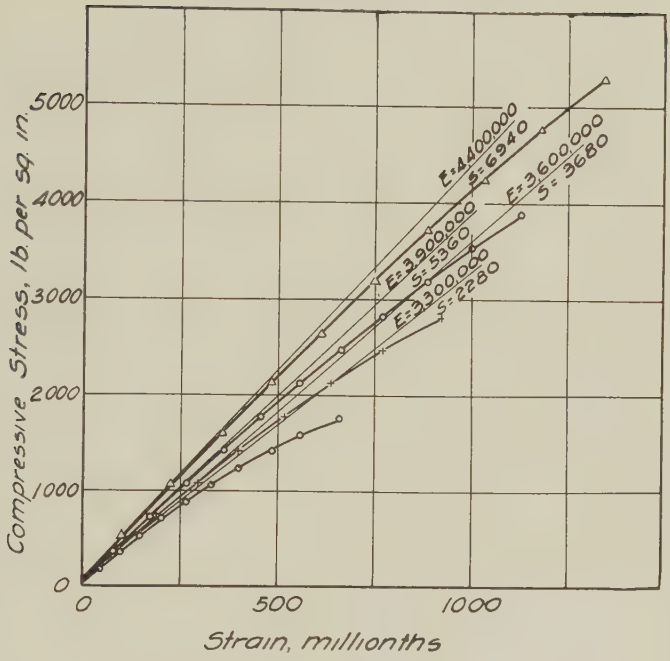


FIG. 20—AVERAGE STRESS-STRAIN DIAGRAM FOR CONTROL CYLINDERS OF SERIES 2

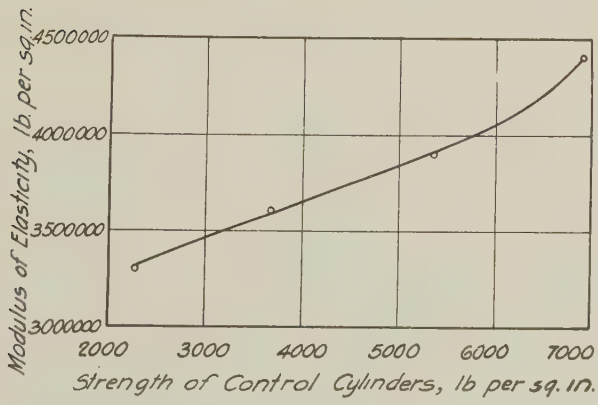


FIG. 21—RELATION BETWEEN MODULUS OF ELASTICITY AND STRENGTH OF CONTROL CYLINDERS FOR SERIES 2

TABLE 6—SUMMARY OF TESTS OF REINFORCEMENT

Nominal Size	No. of Coupons Tested	Yield-Point Stress lb. per sq. in.	Ultimate Strength lb. per sq. in.
Longitudinal Reinforcement—Intermediate Grade			
½-in. square	28	43,560	66,960
½-in. round	6	51,640	76,550
⅝-in. round	6	45,540	73,660
¾-in. round	6	45,470	69,220
		Structural Grade	
½-in. square	24	40,840	64,320
		Rail Grade	
½-in. square	21	65,320	102,680
Spiral Reinforcement—Intermediate Grade			
No. 5 (0.207 diam.)	65	Not Determined	82,760

mination of the modulus of elasticity was satisfactory. The simplicity of this method makes it very convenient for determination of modulus of elasticity where greater accuracy is not possible on account of time and expense involved.

15. *Reinforcement*—The longitudinal reinforcement consisted of intermediate, structural, and rail grades of steel bars. The intermediate and structural grades of bars were donated by the Concrete Steel Company of New York, and the rail steel by the Rail Steel Bar Assocation, Chicago. The bars were 22 ft. long when received. At the laboratory they were cut into four 5 ft. lengths and a 2-ft. test coupon. The four 5 ft. lengths were used in the same column. Four ½-in. diameter bars correspond to 1.5 per cent of longitudinal reinforcement, eight ½-in. square bars correspond to 4 per cent, and four ⅝-in. diameter plus four ¾-in. diameter bars correspond to 6 per cent of reinforcement. The coupons from each set of four bars were tested for yield-point stress and ultimate strength. The summary of these tests is given in Table 6. Both yield-point stress and the ultimate strength of the coupons (based upon nominal areas) varied considerably. The variation in weights of bars of the same size indicates that the variation in strength was probably due to the

variation of the cross-section from the nominal size of the bars, rather than differences in quality of the steel.

The yield point and ultimate strength for the structural grade were only slightly lower than those for the intermediate grade steel. The rail steel showed yield point and ultimate strength considerably higher than those for the intermediate grade. The marked influence of this difference will be pointed out later.

The lateral reinforcement consisted of wire spirals. The spiral wire was furnished by the American Steel and Wire Company, Chicago, and fabricated into spirals by the American System of Reinforcing, Chicago. The spirals had an outside diameter of 8 in., and an overall length of 59 in. Three extra turns were provided at each end of the spiral. A No. 5 wire having a 1.35-in. pitch gave 1.2 per cent lateral reinforcement. A test coupon was attached to each spiral. The average tensile strength on these coupons is given in Table 6. It will be noted that the ultimate strength of the No. 5 wire was considerably above that of the intermediate grade longitudinal reinforcement.

The reinforcement for each column was assembled into one unit before being placed in the mold. For the purpose of carrying the strain gage holes, ½-in. steel cubes were welded to four of the longitudinal bars at intervals of 10 in. along their length, thus forming a continuous row of five 10-in. gage lengths on each bar. The end cubes were placed five inches from the ends of the column.

The spirals had three spacers which were placed approximately 120 deg. apart. The longitudinal bars were fastened to the inside periphery of the spiral by means of wires.

16. *Placing and Curing of Concrete*—The mixing was done in a kettle-shaped mixer. The dry materials were mixed for two minutes, the water was then added, and the mixing was continued for four minutes more. One batch of concrete was sufficient for one column and its three control cylinders. For the plastic concrete continuous placing was used in the concreting of the columns. During the placing, the concrete was rodded continuously with a ½-in. steel rod and the mold was tapped with a

hammer in order to consolidate the mass. The concrete to be placed by the vibrator was placed in three equal layers. Each layer was vibrated for about 45 seconds.

About four hours after concreting, the columns were capped with a remixed neat cement paste. At the age of 24 hours the molds were removed, and a steel plug ¼ in. in diameter and ½ in. long was set in the concrete opposite each lug on the longitudinal bars. Neat cement paste was used to embed them in the concrete. The plugs in the concrete and the lugs on the bars were coated with "Cosmic anti-rust preparation," and the columns were placed in the moist room. At the age of seven days the columns were removed from the moist room, holes were drilled in the plugs and lugs, strain gage observation were taken, and the columns were replaced in the moist room to remain until an age of 56 days was reached.

17. *The Testing*—At the age of 56 days the columns were removed from the moist room and placed in the 800,000-lb. screw-power testing machine. The testing crew consisted of two men, an observer and a recorder. Strain gage readings were taken on the entire 40 gage lengths before applying any load and again at each increment of load. About 10 increments were used. The time required for a full set of observations was about seven minutes. With the "fast" loading the weighing beam was re-balanced after completion of each set of readings and a new increment of load was applied immediately.

With the "slow" loading the procedure was similar to that for fast loading up to about 70 per cent of the maximum load. From that time on four hours elapsed between successive increments of load, and readings were taken immediately after completion of one increment and again just before starting of the next one. During the interval between increments the load was kept within about 2000 lb. of that intended. This was done by "bringing up the load" whenever it had fallen off appreciably. The frequency with which the machine had to be operated to maintain the load varied from a few minutes to more than half an hour within any given period between increments. Near the maximum load the frequency with which it was necessary to bring up the load was much greater than for the lower loads. To prevent drying out

during the four-hour period in which the load was held, the column was enclosed in a canvas jacket which was kept moist by sprinkling. All the columns were photographed after completion of the test. A measure of the reliability of the experimental work of this investigation is found in the close agreement of measured strains (1) between the steel and the concrete of the same column, (2) between companion columns of any given group, (3) between



FIG. 22—A PLAIN CONCRETE COLUMN AFTER FAILURE

two groups of the same kind, one of which was used for fast loading and the other for slow loading. This agreement indicates not only good strain gage work but uniformity of making, capping, and curing of the test specimens.

18. *Plain Columns*—The ultimate strengths of the plain columns and of their control cylinders are given in Table 7, also the strengths of the columns stated as a percentage of the strength of the control cylinders. The average strength of the plain columns was approximately 85 per cent of the strength of the cylinders and the uniformity of both the cylinders and the columns was good. The plane of failure generally made an angle of about 30 deg. with the axis of the column as illustrated in Fig. 22. Failure generally occurred within the upper half of

TABLE 7—STRENGTH OF PLAIN CONCRETE COLUMNS UNDER FAST LOADING AND SLOW LOADING

Strengths are given in pounds per square inch

Method of Loading Columns	Strength of Control Cylinders				Designed Strength
	For Column No.			Average	
	1	2	3		
Fast	2120	2000	2210	2110	2000
Slow	2250	2220	2080	2180	2000
Fast	3000	3600	3420	3340	3500
Slow	3220	3450	3450	3370	3500
Fast	5050	5260	5280	5200	5000
Slow	5130	5120	5480	5240	5000
Fast	7470	7450	7060	7330	8000
Method of Loading Columns	Strength of Columns				Per cent Cylinder Strength
Fast	1820	1850	1680	1780	84.5
Slow	1680	1680	1680	1680	77.8
Fast	2790	3010	3050	2950	88.5
Slow	2800	2520	2800	2710	80.5
Fast	4290	4470	4130	4300	82.8
Slow	4770*	4770*	....†	4770*	91.1*
Fast	6680	5930	7070	6560	89.5
				Av.	85.0

*Accidentally run beyond the last 10 per cent increment.
†Tipped over and broken before the testing.

the column, a fact which is consistent with the observation in Series 1, that the eccentricity of loading was largest near the top.

The columns of 8000-lb. concrete failed explosively. Those with 5000-lb. concrete failed less abruptly but almost without warning. The columns of 3500-lb. concrete failed gradually and in some cases it was possible to get them out of the machine whole. The columns with 2000-lb. concrete settled down gradually under the load and were removed from the machine in one piece.

The stress-strain curves for the cylinders and the columns tested with fast loading are given in Fig. 23, 24, 25, and 26, for

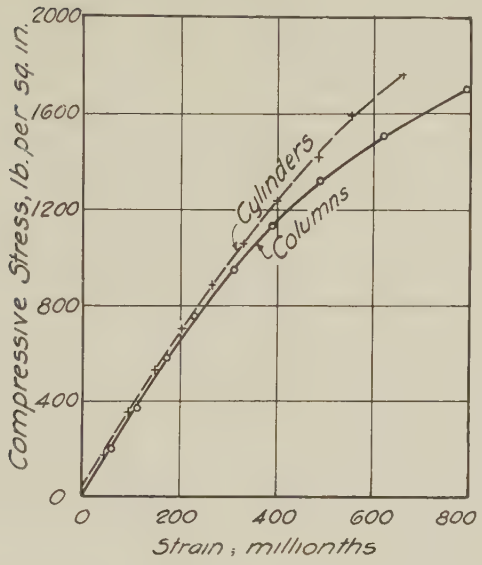


FIG. 23—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 2000 LB. PER SQ. IN.

the four strengths of concrete. Each curve is the average for all the specimens of its kind. The solid lines represent the columns and the dotted lines represent the cylinders. The outstanding features of these figures are the close agreement between the curves for the cylinders and those for the columns within the lower half, and the consistently greater deformation in the columns within the upper half of the curves. Generally the curve for the cylinders lies slightly above that for the columns even at low loads, indicating apparently, that there was some slight delay in getting the strain measuring apparatus used on the cylinders into action.

The modulus of elasticity given by the slope of the stress-strain curves for the columns for a stress of 500 lb. per sq. in. is in close

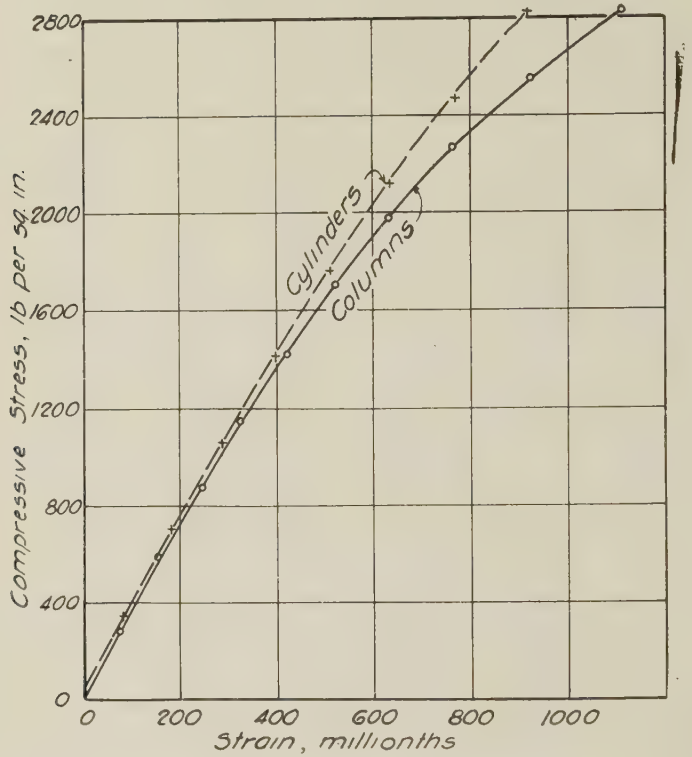


FIG. 24—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 3500 LB. PER SQ. IN.

agreement with that found for the cylinders.

Fig. 27 shows the stress-strain curves for fast and slow loading of columns designed for strengths of 2000, 3500, and 5000 lb. per sq. in. The horizontal portions of the curves represent the deformation which occurred during the four hours under practically constant load. The outstanding feature is the close agreement of those portions of the load strain curves in which the load was applied rapidly.

The horizontal distance between the upper parts of the two curves for the same concrete strength represents the portion of

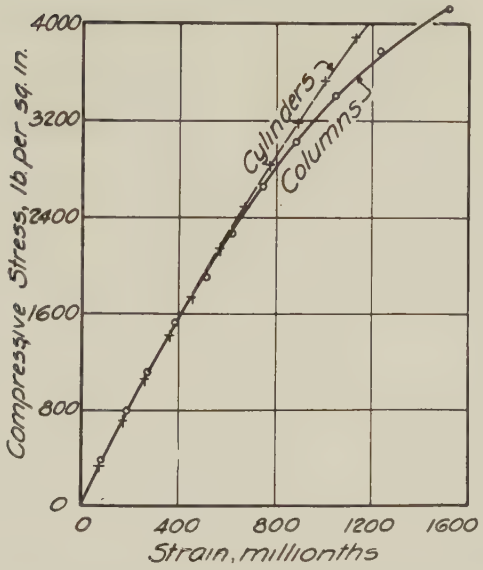


FIG. 25—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 5000 LB. PER SQ. IN

the total deformation which is due to the holding of the load for four hours.

19. *Relation Between Strains and Percentage of Longitudinal Reinforcement*—Figs. 28 and 29 show the stress-strain relations for columns of concrete designed for strengths of 8000 and 3500 lb. per sq. in. respectively. The spiral reinforcement amounted to 1.2 per cent for all the columns, and the longitudinal reinforcement varied from 0 to 6 per cent as indicated. The stress $\frac{P}{A}$ plotted in these diagrams is the load on the column divided by the total area within the outer circumference of the spiral, that is, by 50.3 sq. in., the sum of the sectional areas of the circumscribed concrete and the longitudinal reinforcement. The portion of this average stress that was considered to be carried by the longitudinal reinforcement is indicated by the broken straight lines in the lower part of the diagram. In constructing

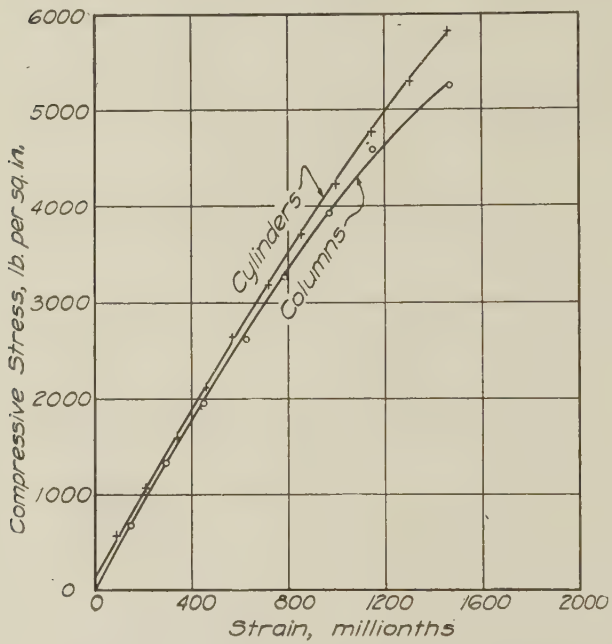


FIG. 26—STRESS-STRAIN DIAGRAM FOR PLAIN COLUMNS AND FOR CONTROL CYLINDERS; DESIGNED CONCRETE STRENGTH 8000 LB. PER SQ. IN.

these lower lines the modulus of elasticity of the steel was taken at 30,000,000 lb. per sq. in. and the stresses were determined from the measured strains. The break in each line comes at the strain corresponding to the yield-point stress of the bars of the size used to secure the percentage of reinforcement indicated.

The stress-strain curves for the control cylinders made with the columns having only spiral reinforcement, have been plotted in Figs. 28 and 29 also. In each figure the curve for the cylinder is a dotted line and its agreement with the curve for the corresponding column is very good. Figs. 28 and 29 indicate much less departure of the strain in the spiraled columns from that in their control cylinders than is shown in Fig. 23 and 26 for the plain columns. The indication is that the presence of the spiral had a slight effect in reducing the longitudinal strain in the

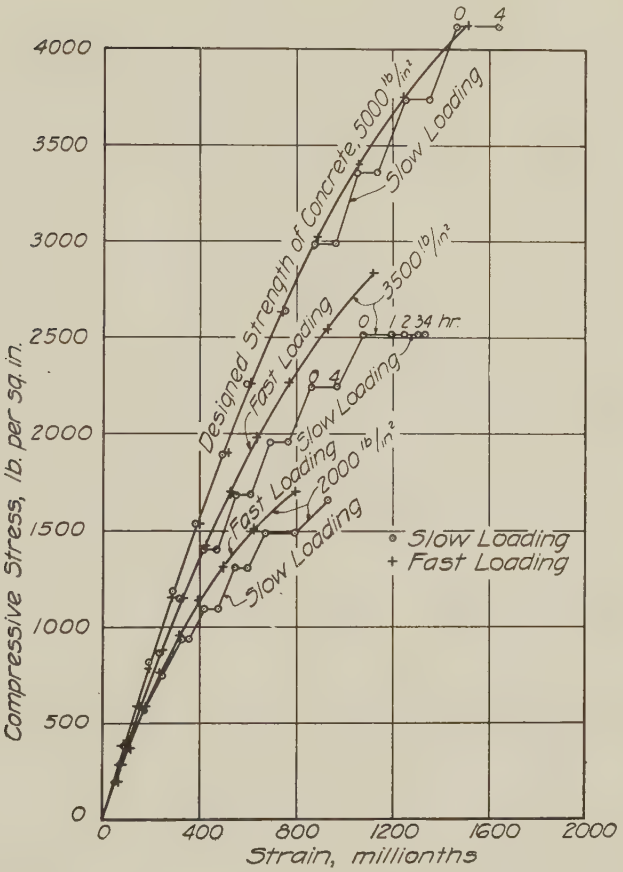


FIG. 27—STRESS-STRAIN DIAGRAM FOR PLAIN CONCRETE COLUMNS, FAST AND SLOW LOADING

columns for stresses above about one-half the cylinder strength, but no effect for lower stresses.

For the concrete designed for a compressive strength of 8000 lb. per sq. in., Fig. 28, the vertical distances between the stress-strain curves for the columns with various percentages of longitudinal reinforcement are very nearly the same as the vertical distances between the corresponding straight lines representing

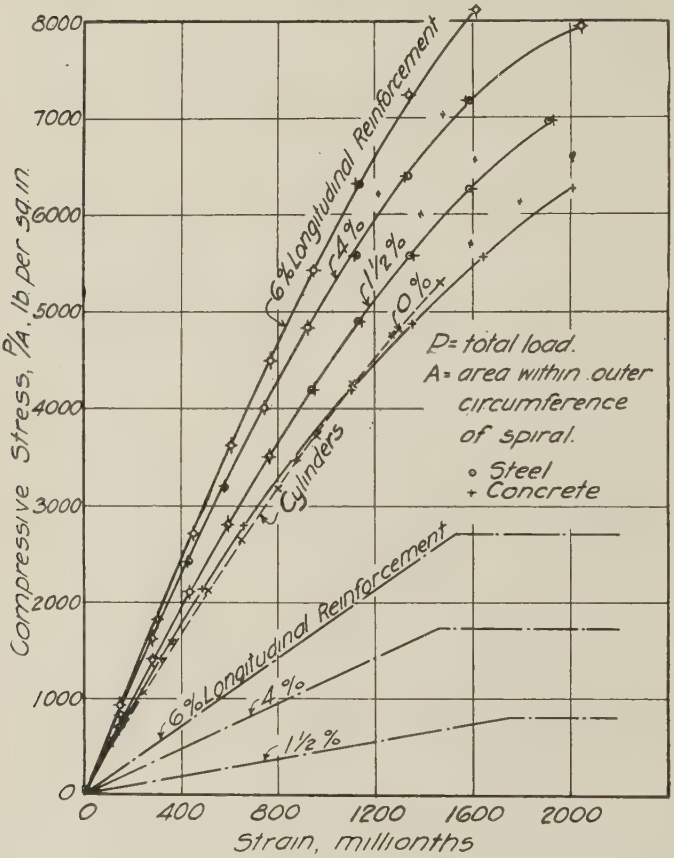


FIG. 28—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT; DESIGNED CONCRETE STRENGTH 8000 LB. PER SQ. IN.; 1.2 PER CENT SPIRAL REINFORCEMENT

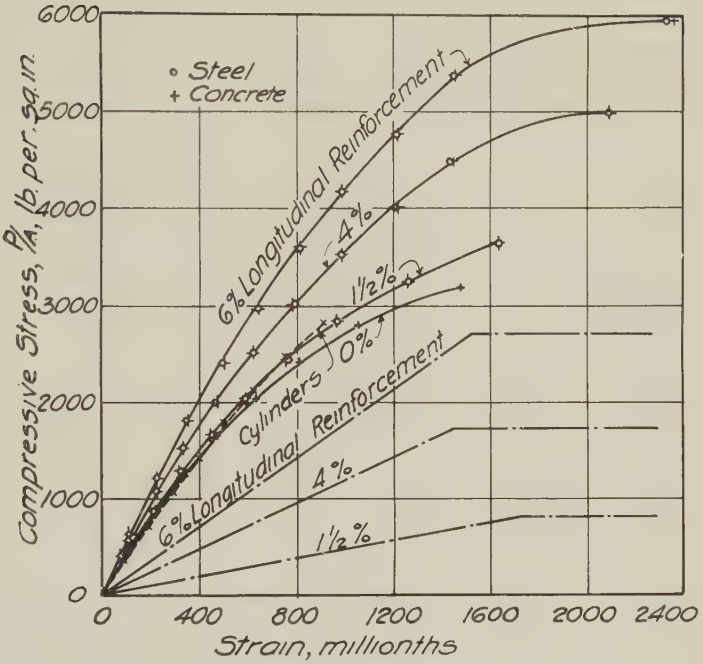


FIG. 29—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT; DESIGNED CONCRETE STRENGTH 3500 LB. PER SQ. IN.; 1.2 PER CENT SPIRAL REINFORCEMENT, FAST LOADING

the stresses carried by the reinforcement. That is, if the ordinates to the straight lines (the stresses carried by the longitudinal reinforcement) were subtracted from the ordinates to the stress-strain curves for the columns, the resulting curves would practically coincide with the curve for the column having no longitudinal reinforcement. This indicates that the presence of longitudinal reinforcement had no perceptible effect on the elastic behavior of the concrete designed for a strength of 8000 lb. per sq. in.

With the 3500-lb. concrete shown in Fig. 29 for fast loading and Fig. 30 for slow loading, the case was different. The vertical distances between the stress-strain curves for 0 and 1½ per cent

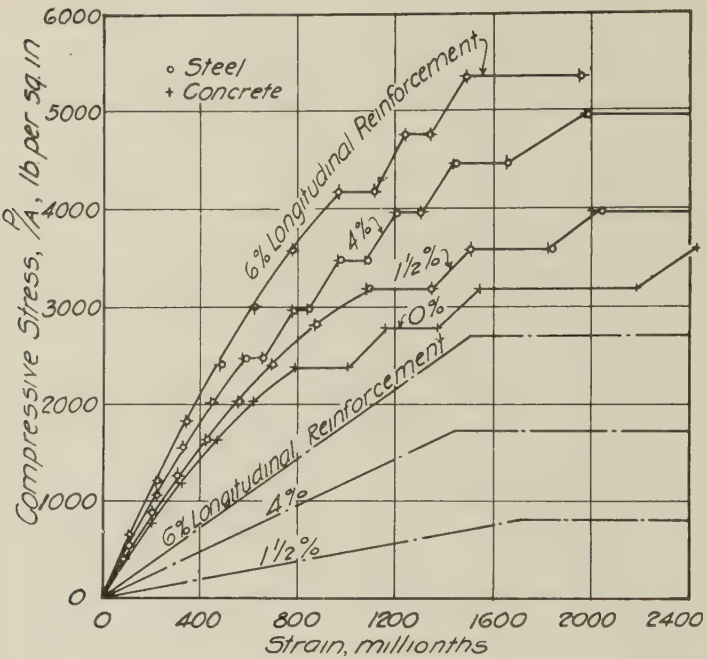


FIG. 30—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT; DESIGNED CONCRETE STRENGTH 3500 LB. PER SQ. IN.; 1.2 PER CENT SPIRAL REINFORCEMENT, SLOW LOADING

reinforcement were considerably less than the ordinates of the line representing the stresses carried by 1½ per cent of reinforcement. With this exception, the conditions for the 3500-lb. concrete were about the same as those for the 8000-lb. concrete with the result that if the stresses carried by the reinforcement were subtracted from the total average stresses carried by the corresponding columns the resulting curves for the stresses carried by the concrete for the reinforced columns would practically coincide with each other, but would lie below the curve for the columns without longitudinal reinforcement. The behavior is as though the introduction of the longitudinal reinforcement reduced the effective sectional area of the concrete by a constant amount in

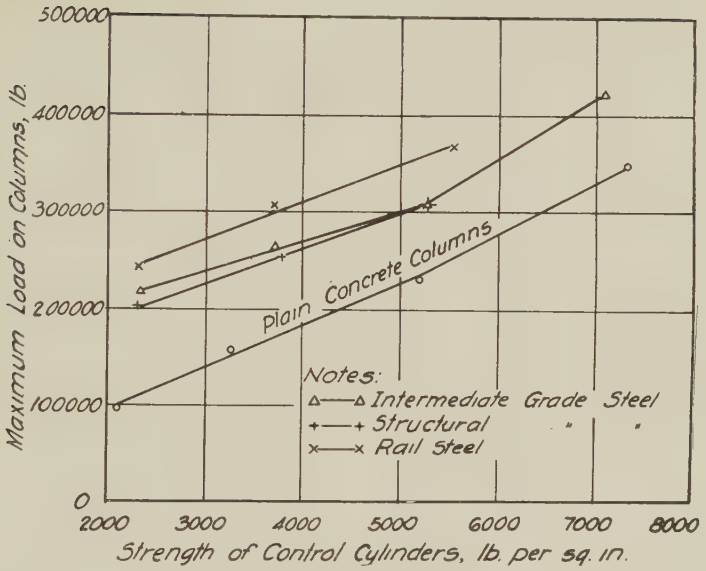


FIG. 31—COMPARISON OF PLAIN CONCRETE COLUMNS WITH COLUMNS HAVING 4 PER CENT LONGITUDINAL REINFORCEMENT OF THREE DIFFERENT GRADES, 1.2 PER CENT SPIRAL REINFORCEMENT OF INTERMEDIATE GRADE

excess of the area of the reinforcement used. The only explanation of the difference in behavior between the 8000 and 3500-lb. concrete which seems reasonable is based upon a possible difference in compactness due to the difference in the methods of placing. The 8000-lb. concrete was vibrated during the placing, and the 3500-lb. concrete was not.

20. *Relation Between Strength of Column and Strength of Concrete*—Fig. 31 shows, as ordinates, the total loads carried by the columns and as abscissas, the strengths of their control cylinders. The lower curve represents the plain columns and the upper curves represent columns having 4 per cent longitudinal and 1.2 per cent spiral reinforcement. The spiral reinforcement was all of intermediate grade; the longitudinal included structural,

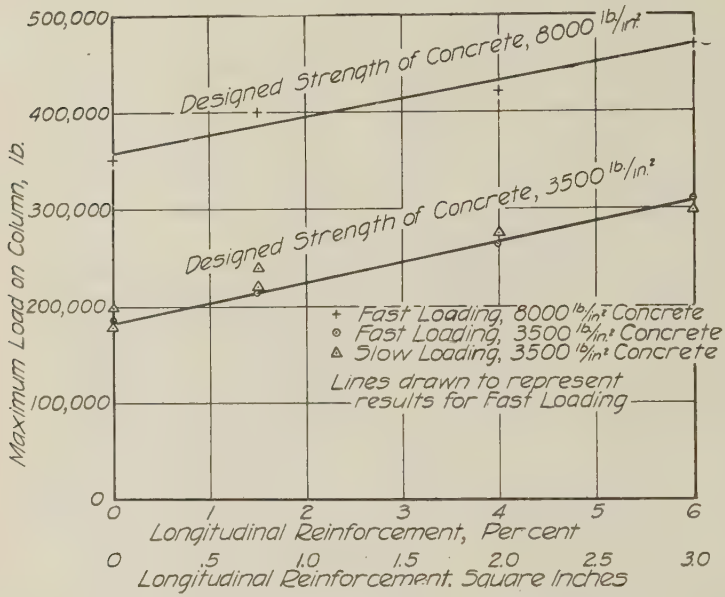


FIG. 32—EFFECT OF VARIATION IN PERCENTAGE OF LONGITUDINAL REINFOFCEMENT ON THE STRENGTH OF THE COLUMNS, 1.2 PER CENT SPIPAL REINFORCEMENT

intermediate, and rail grades of steel. The strengths of the columns increased quite consistently with the increase in strength of the control cylinders. The rate of increase in strength of the reinforced columns seems to have been nearly equal to that of the plain columns when net core areas* of concrete in the reinforced columns are considered. That is, the increase in the strength of the concrete was just as effective in a reinforced column as it was in a plain column. It is shown in a previous section that for a plain column the strength was 85 per cent of its cylinder strength regardless of the strength of the concrete. Therefore the increase in strength of the reinforced columns would be equal to the net area of its concrete times 85 per cent of the increase in strength of its control cylinders, regardless of the grade of longitudinal reinforcement.

*By the term "net core area" of the concrete is meant the area within the outer circumference of the spiral minus the sectional area of the longitudinal reinforcement.

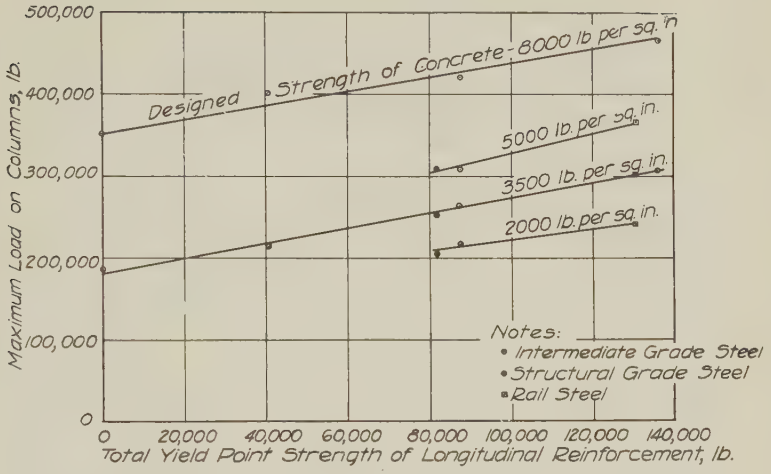


FIG. 33—RELATION BETWEEN TOTAL YIELD-POINT STRENGTH OF LONGITUDINAL REINFORCEMENT AND STRENGTH OF COLUMN, 1.2 PER CENT SPIRAL REINFORCEMENT

In Fig. 32 the maximum loads carried by reinforced columns have been plotted as ordinates and the percentage of longitudinal reinforcement as abscissas. The percentage of spiral was 1.2 for all columns. The straight line representing the relation between the strength of the column and the percentage of reinforcement for a concrete strength of 3500 lb. per sq. in. is parallel to the line representing the same relation for the 8000-lb. concrete. That is, the strength added to a reinforced column due to a given increase in the concrete strength was the same for all the percentages of longitudinal reinforcement used.

Fig. 33 shows the relation between the strength of the columns and the total yield-point strength of the longitudinal reinforcement. This figure is a combination of the previous two figures and shows that the increase in strength of columns due to increased strength of the concrete was practically independent of the yield point strength of the reinforcement used in the columns.

The effect of the strength of the concrete upon the deformation of columns having 4 per cent longitudinal and 1.2 per cent spiral

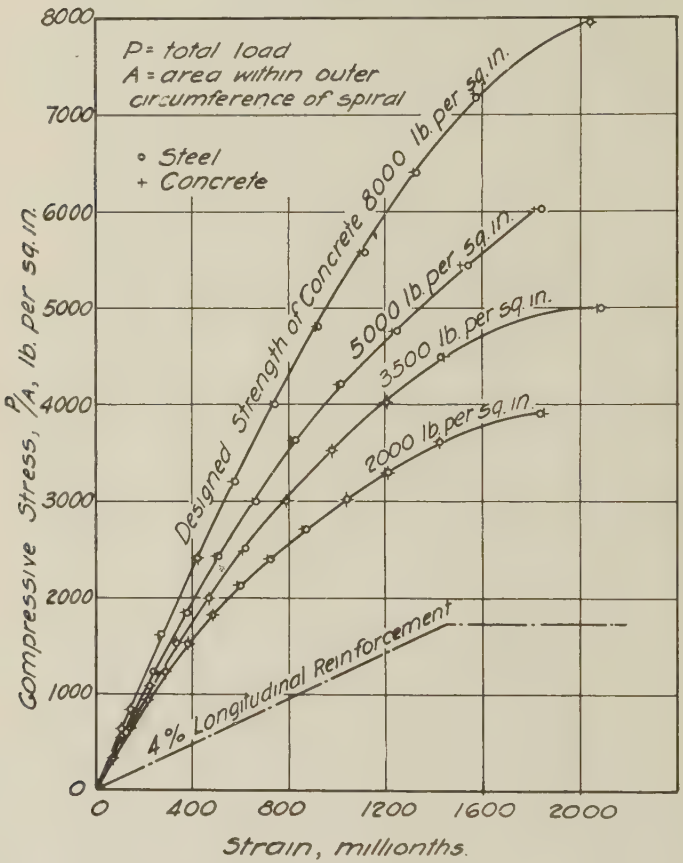


FIG. 34—STRESS-STRAIN DIAGRAM FOR COLUMNS WITH DIFFERENT CONCRETE STRENGTHS, REINFORCEMENT 4 PER CENT LONGITUDINAL AND 1.2 PER CENT SPIRAL

reinforcement is illustrated in Fig. 34. The strains for a given stress decreased quite regularly with the increase in strength of the concrete.

21. *Relation Between Yield-Point Strength of Longitudinal Reinforcement and Strength of Column*—The strength added by the longitudinal reinforcement in the columns may be varied by changing either the percentage or the grade of reinforcing steel.

The effect of various grades of steel upon the strength of columns of a given percentage of reinforcement and strength of concrete is illustrated in Fig. 31. Intermediate, structural, and rail grades were included in these tests, and the column strengths obtained with these grades of longitudinal reinforcement are shown in the three upper lines. The yield-point strength of the reinforcement is given in Table 6. Structural and intermediate grades had nearly equal yield-point stresses (40,840 and 43,560 lb. per sq. in.), but the rail grade had a much higher yield-point stress (65,320 lb. per sq. in.). The lines representing the strengths of the columns having structural and intermediate grades of reinforcement are close together, while the line representing the strength for the columns with rail grade reinforcement lies considerably higher than the lines for the other grades.

In Fig. 33 the strengths of the columns have been plotted as ordinates and the total yield-point strength of the longitudinal steel as abscissas. Straight lines are obtained for each strength of concrete. The increase in yield-point strength was due in some cases to change in grade of steel, and in other cases to increased percentage of reinforcement. Since for a given strength of concrete, all points group themselves along the same line, the strength of the column seemed to be independent of whether the change in yield-point strength was affected by varying the percentage of a given grade of reinforcement or by using different grades of reinforcement. The lines for different strengths of concrete are practically parallel, indicating that the increase in strength due to the higher yield-point strength of the reinforcement was independent of the strength of the concrete. The slope of these lines represents the strength added to the columns for each unit of increase in strength of the reinforcement. The slope is, therefore, a measure of the effectiveness of the longitudinal reinforcement in adding strength to a column of a given concrete strength. The average slope of the curves in Fig. 33 indicate that the reinforcement added 97 per cent of its yield-point strength to the strength of the concrete of the core. The yield-point stress of the steel was determined by the "drop of beam," a method which tends to give values slightly too high. The effectiveness of the longitudinal reinforcement may therefore be

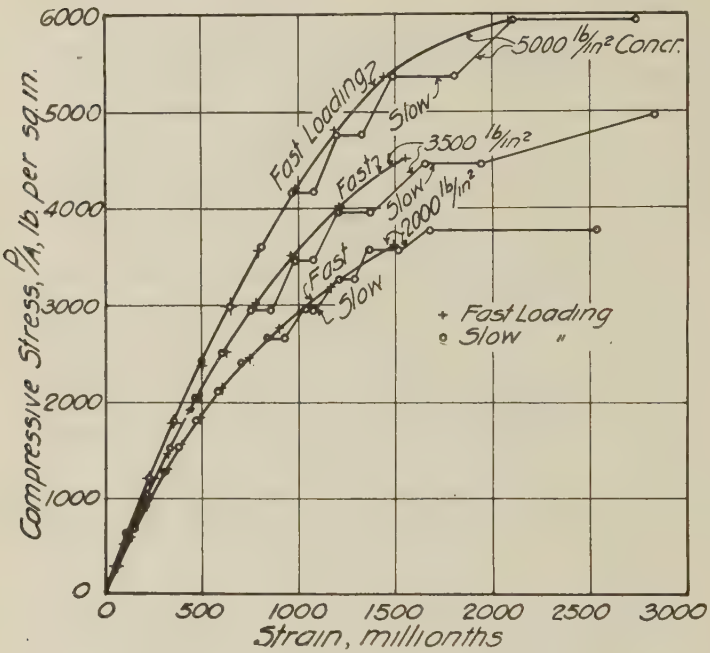


FIG. 35—STRESS-STRAIN DIAGRAM FOR COLUMNS TESTED UNDER FAST LOADING AND SLOW LOADING; 4 PER CENT LONGITUDINAL REINFORCEMENT OF STRUCTURAL GRADE, 1.2 PER CENT SPIRAL REINFORCEMENT

more than 97 per cent of its yield-point strength. In the further discussion in this report the longitudinal reinforcement has been assumed to add its full yield-point strength to the strength of the column. That is, it is considered 100 per cent effective.

22. *Effect of Rate of Loading on Deformation and Strength of Columns*—In Fig. 31 it is shown that the reinforced columns continued to deform during the 4-hour loading intervals. Up to the first load which was maintained for four hours, the deformations of the two sets of columns (designed for "fast" and "slow" loading) were practically equal in all cases, indicating good uniformity in the two sets of columns. The deformation during the first 4-hour interval was generally quite large. The deformation during the succeeding 4-hour intervals was not much different

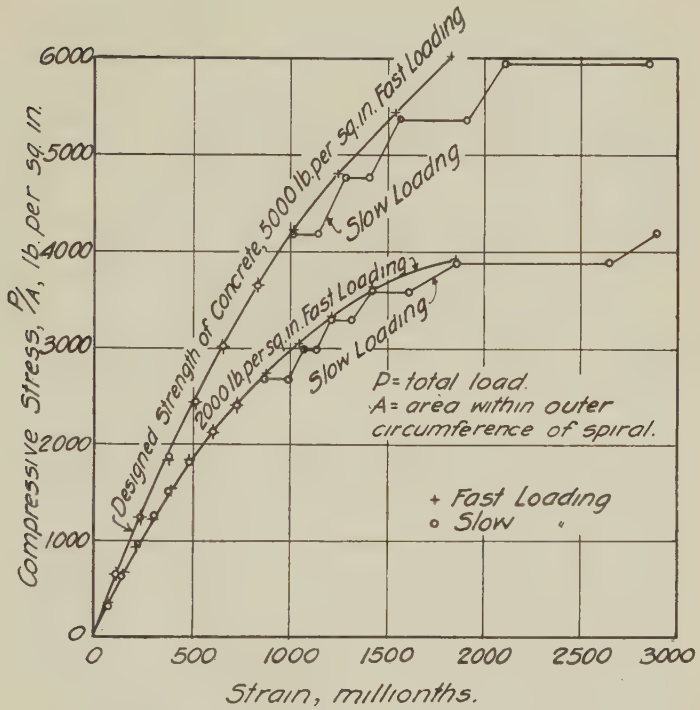


FIG. 36—STRESS-STRAIN DIAGRAM FOR COLUMNS TESTED UNDER FAST LOADING AND SLOW LOADING; 4 PER CENT LONGITUDINAL REINFORCEMENT OF INTEFMEDIATE GRADE, 1.2 PER CENT SPIRAL REINFORCEMENT

from the first one except near the maximum load, when the deformations became several times as large as during the previous intervals. During the application of an increment of load, the rate of deformation was so much lower than that for fast loading that the stress-strain curve was brought very nearly back to the curve for the fast loading.

Figs. 35, 36, and 37 show the deformation for fast and slow loading on columns reinforced with structural, intermediate, and rail grade longitudinal reinforcement. The grade of reinforcement had no effect upon the deformation up to a load near the yield-point strength of the steel. The agreement between de-

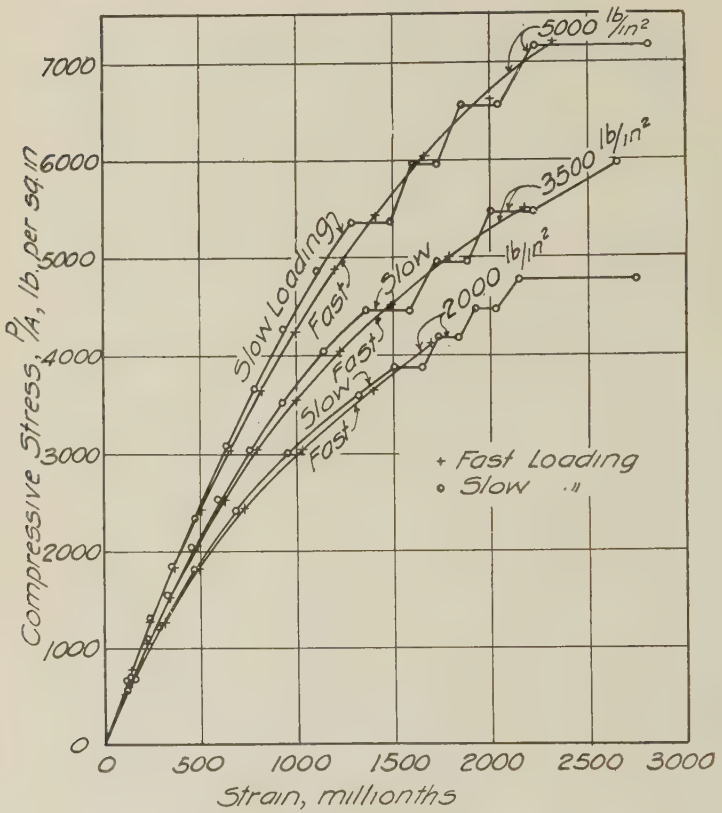


FIG. 37—STRESS-STRAIN DIAGRAM FOR COLUMNS TESTED UNDER FAST LOADING AND SLOW LOADING; 4 PER CENT LONGITUDINAL REINFORCEMENT OF RAIL GRADE, 1.2 PER CENT SPIRAL REINFORCEMENT

formations at low loads for the two methods of testing is evident. In Fig. 37, showing the deformation of the columns having rail steel, the agreement between deformations found for fast and slow loadings was not so good as in the other cases.

The strengths of the columns tested with fast loading and slow loading are given in Tables 7 to 10. Table 7 gives the results for plain columns, and Tables 8, 9, and 10 for reinforced columns. The plain columns tested with slow loading generally developed less strength than did the companion columns with fast loading.

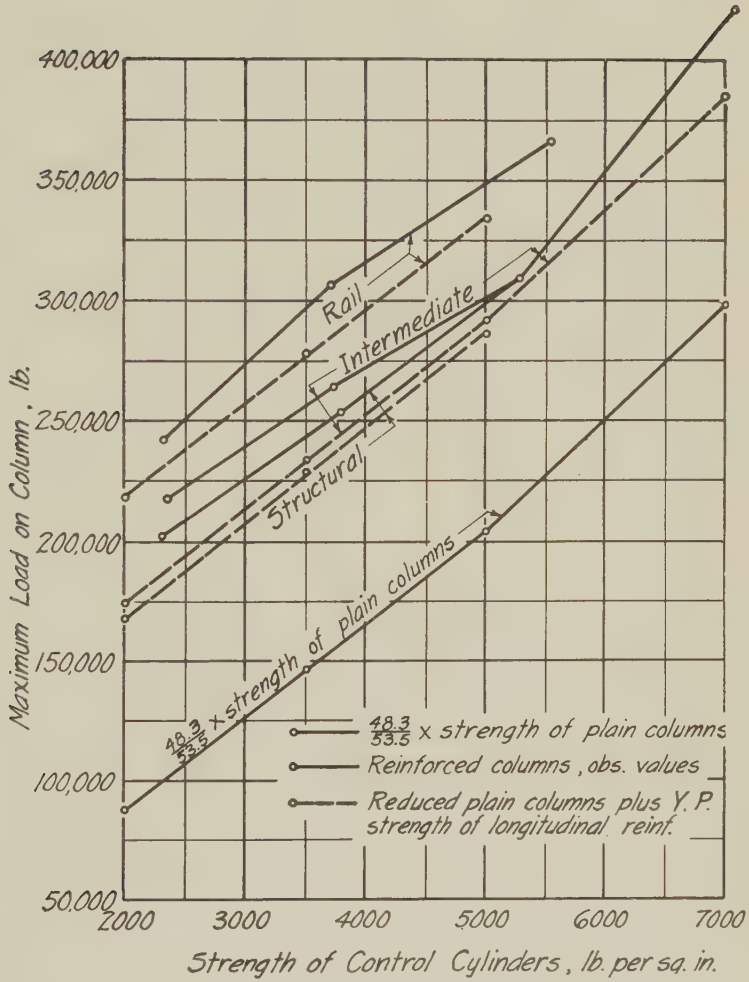


FIG. 38—DIVISION OF LOAD BETWEEN CONCRETE AND REINFORCEMENT

The difference, however, was not very marked. For the reinforced columns the strength obtained with slow loading was very nearly equal to, but generally larger, than the strength obtained with fast loading.

23. *Division of Load Between Concrete and Reinforcement*—Fig. 38 is designed to show the relation between the strengths of the reinforced columns and the sum of the strengths of the concrete and the reinforcement used. All the reinforced columns used in this figure had 4 per cent longitudinal and 1.2 per cent spiral reinforcement. The sectional area of the concrete within the outer circumference of the spiral, that is, the core area minus the sectional area of the longitudinal bars was 48.3 sq. in., and the sectional area of the plain columns was 53.5 sq. in. The plotted values in the lower curve are $\frac{48.3}{53.5}$ times the strength of the plain columns, and represent the strength of the concrete in the reinforced columns. To these strengths have been added the total yield-point strengths of the longitudinal reinforcement. The strengths thus obtained for columns reinforced with structural, intermediate, and rail grades of reinforcement are shown as three distinct dotted lines. In addition, the strengths of the columns so reinforced but having in addition 1.2 per cent spiral reinforcement are shown as solid lines. Based upon the assumption that the longitudinal reinforcement added its full yield-point strength to the column (as indicated in section 21), the vertical distance between the solid and the dotted lines for each grade of steel represents the strength added by 1.2 per cent of spiral reinforcement. The solid line and the dotted line are not far from parallel, indicating that nearly a constant strength was added to the column by the spiral regardless of the strength of the concrete. The average total added strengths so determined were 16,700, 19,000, and 16,000 lb. for longitudinal reinforcement of structural, intermediate, and rail grades of steel respectively. Apparently the strength added by the spiral was not affected by variations in the yield-point strength of the longitudinal reinforcement. The average of the added strengths for the three grades was 17,200 lb. for 1.2 per cent of spiral, or 14,300 lb. for 1

TABLE 8—EFFECT OF GRADE OF LONGITUDINAL REINFORCEMENT ON STRENGTH OF COLUMNS

Reinforcement in all columns: 4 per cent longitudinal and 1.2 per cent spiral.

Concrete Strength				Load on Column at Failure			Method of Loading
Designed	Obtained			1	2	Av.	
	1	2	Av.				
Intermediate Grade							
2000	2360	2340	2350	221,000	214,200	217,600	Fast
2000	2360	2400	2380	*1.5 hr. 210,000	3.5 hr. 210,000	2.5 hr. 210,000	Slow
3500	3790	3660	3730	260,500	268,000	264,250	Fast
3500	3810	3980	3900	*15 min. 275,000	15 min. 275,000	15 min. 275,000	Slow
5000	5150	5410	5280	321,000	297,500	309,250	Fast
5000	5160	5670	5420	* 0 330,000	5 min. 300,000	—	Slow
Structural Grade							
2000	2250	2360	2310	201,500	202,700	202,100	Fast
2000	2320	2370	2350	*5 min. 210,000	40 min. 210,000	25 min. 210,000	Slow
3500	3750	3810	3780	252,600	253,000	252,800	Fast
3500	3830	3800	3820	*25 min. 225,000	30 min. 250,000	—	Slow
5000	5180	5350	5270	311,500	308,000	309,750	Fast
5000	5270	5480	5380	* 0 330,000	45 min. 300,000	—	Slow
Rail Grade							
2000	2380	2230	2310	239,300	244,700	242,000	Fast
2000	2260	2520	2390	max. 260,200	*7 min. 255,000	—	Slow
3500	3650	3740	3700	307,000	304,700	305,850	Fast
3500	3800	3780	3790	*25 min. 300,000	0 325,000	—	Slow
5000	5460	5620	5540	355,000	378,000	366,750	Fast
5000	5530	5800	5670	*0 390,000	0 390,000	0 min. 390,000	Slow

*Time from application of maximum load until failure.

TABLE 9—EFFECT OF VARIATION IN PERCENTAGE OF LONGITUDINAL REINFORCEMENT ON STRENGTH OF COLUMNS

Fast Loading

Per cent Reinforcement		Cylinder Strength	Maximum Load at Failure	Assumed Load Distribution		Nominal Area of Long. Rein.
Longi-tudinal	Spiral			Concrete 85% of Cyl. Strength	Attributed to Presence of Reinforcement	
		lb. per sq. in.	lb.	lb.	lb.	sq. in.
Designed Cylinder Strength, 8000 lb. per sq. in.						
0	0	7330	348,300	348,300	0	0
0	1.2	6590	350,400	282,000	68,400	0.
1½	1.2	6600	400,150	278,000	122,150	0.785
4	1.2	7070	421,500	290,000	131,500	2.00
6	1.2	6950	470,000	280,000	190,000	2.99
Designed Cylinder Strength, 3500 lb. per sq. in.						
0	0	3340	157,900	157,900	0	0
0	1.2	3760	185,700	160,700	25,000	0
1½	1.2	3410	213,900	143,500	70,400	0.785
4	1.2	3730	264,250	153,000	111,250	2.00
6	1.2	3770	308,450	151,450	157,000	2.99

TABLE 10—EFFECT OF VARIATION IN PERCENTAGE OF LONGITUDINAL REINFORCEMENT ON STRENGTH OF COLUMNS

Designed strength of concrete—3500 lb. per sq. in.
Spiral reinforcement—1.2 per cent

Per cent Longitudinal Reinforcement	Concrete Strength Obtained lb. per sq. in.			Load on Columns lb.			Method of Loading
	1	2	Av.	1	2	Av.	
0.0	3790	3730	3760	186,900	184,500	185,700	Fast
0.0	3920	3810	3870	35 min. 200,000	85 min. 180,000	—	Slow
1.5	3710	3100	3410	218,500	209,300	213,900	Fast
1.5	3920	3800	3860	15 min. 220,000	0 240,000	—	Slow
4.0	3790	3660	3730	260,500	268,000	264,250	Fast
4.0	3810	3980	3900	15 min. 275,000	15 min. 275,000	15 min. 275,000	Slow
6.0	3850	3680	3770	305,000	311,900	308,450	Fast
6.0	3690	3790	3740	15 min. 300,000	5 min. 300,000	10 min. 300,000	Slow

per cent. This is 70 per cent of the average strength added by 1 per cent of longitudinal reinforcement of intermediate grade as shown in Fig. 31.

The indication of the foregoing paragraphs is that the strength of the columns having longitudinal and spiral reinforcement was made up of (1) 85 per cent of the cylinder strength times the net area of the concrete within the outer circumference of the spiral, (2) the yield-point stress of the longitudinal bars times their area, and (3) the strength added by the spiral, which amounted to about 70 per cent of the strength added by an equal percentage of longitudinal reinforcement of intermediate grade steel.

24. *Summary of Results*—With the water content of the concrete constant at 39 gal. per cu. yd. the strength of the concrete increased in direct proportion to the increase in the cement content.

For all except the vibrated concrete, the increase in the modulus of elasticity was very nearly proportional to the increase in the compressive strength.

The strains measured on the steel agreed very well with those measured on the concrete for all columns, and the agreement between strains measured on columns of the same kind was good.

The average strength of the plain columns was approximately 85 per cent of the strength of the cylinders and the uniformity of both the cylinders and the columns was good.

The stress-strain curves for the plain columns practically coincided with those for the corresponding control cylinders up to about one-half the maximum load. For higher loads the strains in the columns were somewhat greater than those in the cylinders.

For the 8000-lb. concrete the load at any strain for columns with different percentages of longitudinal reinforcement was equal to the load carried by a column with no longitudinal reinforcement plus the load carried by longitudinal reinforcement having a modulus of elasticity of 30,000,000 lb. per sq. in. With the 3500-lb. concrete the conditions were the same as for the 8000-lb. concrete except that for any given strain the loads for the columns with all the different percentages of reinforcement were a constant amount less than the sum of the load for the column

without longitudinal reinforcement and the load found to be carried by the reinforcement.

For any two columns reinforced in the same manner, the difference in total strength was approximately equal to the difference in strengths of the concrete in the core as determined by the tests of the plain columns.

The increase in total strength due to the use of longitudinal reinforcement was approximately equal to the yield-point stress of the reinforcement times its sectional area, regardless of the grade of reinforcement used.

The strength of the plain columns tested with slow loading was generally slightly less than that of the columns tested with fast loading. For the reinforced columns the strength with slow loading was nearly equal to, but generally slightly greater than that for fast loading.

The indication of the tests is that the strength of the columns having longitudinal and spiral reinforcement was made up of (1) 85 per cent of the cylinder strength times the net area of the concrete within the outer circumference of the spiral, (2) the yield-point stress of the longitudinal bars times their area, and (3) the strength added by the spiral, which amounted to about 70 per cent of the strength added by an equal amount of longitudinal reinforcement of intermediate grade steel.

Progress Report on Column Tests at the University of Illinois

BY F. E. RICHART* AND G. C. STAEHLE†

INTRODUCTION

The Investigation—The following progress report gives results of tests made in the investigation of reinforced concrete columns at the University of Illinois. As stated in a previous report, the investigation is sponsored by Committee 105 of the American Concrete Institute and the experimental work has been divided between Fritz Engineering Laboratory at Lehigh University and the Materials Testing Laboratory at the University of Illinois. The work at the latter laboratory is under the administrative direction of Dean M. S. Ketchum, Director of the Engineering Experiment Station and Prof. M. L. Enger, Head of the Department of Theoretical and Applied Mechanics.

The testing program outlined by the Committee and reported a year ago[1] has been followed quite closely, variations from it consisting principally of the addition of a few columns of plain concrete, and a small group, Series 8, to bring out the effect of long-continued loading on columns in which the reinforcing bars do not end flush with the end of the column. The time schedule followed in the testing program is given in Table 1.

This report contains results of the tests of Series 1 and 2, in addition to general information pertaining to all of the tests. The data of Series 3, in which the columns are being held under design load for one year before being tested to failure, are naturally incomplete; however, the strains measured over a five-months period of continued loading were felt to be of sufficient interest to warrant an early progress report, to appear in the March

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[1]See Progress Report of Committee 105, Proc. A. C. I. Vol. 26, p. 601, 1930.

TABLE 1—TIME SCHEDULE OF TESTING PROGRAM

All dates given are for 1930 unless otherwise noted and are inclusive.

Operation	Series Number						
	1	2	3	5	6	7	8
Making columns	Jan. Feb.	April Aug.	April July	June Aug.	July Aug.	Aug. Dec.	Dec. Jan. '31
Applying continuous design load			June Sept.				Jan. '31 Feb. '31
Shrinkage and flow measurements			June Sept. '31				Jan. '31 Feb. '32
Loading to failure	Feb. Mar.	June Oct.	June '31 Sept. '31	Aug. Oct.	Sept. Oct.	Oct. Feb. '31	Jan. '32 Feb. '32
Number of columns	10	104	153	20	14	20	12

TABLE 2—TENSILE STRENGTH OF MORTAR BRIQUETS

Made with Lehigh Portland Cement, Standard Ottawa sand, 1:3 by weight.

Date of Making, 1930	Tensile Strength, lb. per sq. in.	
	Age, 7 Days	Age, 28 Days
March 4	316	408
April 25	261	318
June 3	314	376
June 25	294	372
September 5	316	398
Average	300	374

TABLE 3—SIEVE ANALYSES OF AGGREGATES

Values given are average of several analyses

Aggregate	Percentage of sample passing Sieve No.									Fineness Modulus
	100	50	30	16	8	4	⅜″	¾″	1½″	
Sand	3	15	51	73	88	98	100			2.72
Gravel			0	1	1	1	34	78	100	6.85

JOURNAL of the Institute. A similar report will be prepared soon for Series 5 and 6, while the tests of Series 7 and 8 are still unfinished.

1. *Type of Test Column*—All of the columns tested (except two of Series 1) were cylindrical, with a length seven and one-half times the core diameter. In all but those of Series 6, the fireproofing was omitted and the spiral reinforcement was practically at the column surface. In Series 6, a 2-in. thickness of fireproofing was used. As the result of the tests of Series 1, all columns except those of Series 8 were made with the milled ends of reinforcing bars flush with the concrete bearing faces. In Series 8, the ends of bars were stopped ¼ in. short of the end of the column, so that no bearing was transmitted to the bars.

2. *Materials*—The cement used in making columns was furnished by the Lehigh Portland Cement Co. It was stored in wooden barrels in a dry room until used. The strength, as shown by mortar and concrete tests, was uniformly high. Table 2 gives data of briquet tests made at different times during the season. From other tests made from time to time, the water required for normal consistency of the cement was found to be 23.0 to 24.0 per cent, by weight; the average time of initial set was 3 hr. 40 min.; of final set, 6 hr. 40 min. Tests of fineness showed 12 to 14 per cent retained on a No. 200 sieve after 10 min. in a Rotap sieve shaker. All soundness tests gave satisfactory results.

The torpedo sand used was furnished by the Lincoln Sand and Gravel Co., Lincoln, Illinois, and the gravel was furnished by the Neal Sand and Gravel Co., Mattoon, Illinois. Average sieve analysis and fineness moduli are given in Table 3. The aggregates used in Series 1, which differed from the above, are described in the section referring to that series.

The concrete mixtures used were designed to produce cylinder strengths of 2000, 5000 and 8000 lb. per sq. in. at 56 days, except in Series 1, wherein the tests were made at 28 days. A workability as shown by a slump of 3 to 6 in. was chosen, except in the case of the 8000-lb. concrete, which was designed to be placed with the aid of a high frequency electric vibrator, loaned for the work by the Electric Tamper and Equipment Co., Chicago, Ill. The proportions finally used are given in Table 4. The results of

TABLE 4—DATA OF CONCRETE MIXTURES

Materials: Lehigh portland cement; Lincoln, Ill., sand; Attica, Ind., gravel. Unit weight of aggregates, A. S. T. M. method, dry, rodded: sand 110 lb. per cu. ft., gravel 100 lb. per cu. ft.

Designed Strength lb. per sq. in.	Series Number	Proportions by Weight Cement, Sand, Gravel	Water-Cement Ratio	
			By Volume	Gallons per Sack
2000	2, 3, 7	1 : 4.05 : 4.95	1.40	10.5
		1 : 4.05 : 4.95	1.40	10.5
	5, 6	1 : 3.83 : 4.67	1.40	10.5
		1 : 4.18 : 5.27*	1.50*	11.25*
3500	2, 3, 7	1 : 3.27 : 3.98	1.10	8.25
	5, 6	1 : 3.27 : 3.98	1.10	8.25
		1 : 3.15 : 3.85	1.10	8.25
5000	2, 3, 7	1 : 2.25 : 2.75	0.80	6.0
8000	2	1 : 1.01 : 1.24	0.50	3.75

*Used in 28 and 32-inch columns to reduce ultimate column strength.

56-day tests of companion cylinders made with all columns are reported with the results of the various column tests.

The reinforcing bars were furnished by the Kalman Steel Co., Chicago, Ill., the Truscon Steel Co., Youngstown, Ohio, and the Rail Steel Bar Association, Chicago, Ill. The spiral stock was furnished by the American Steel and Wire Co., Chicago, Ill., and fabricated into spirals by the American System of Reinforcing, of Chicago. While complete tests of reinforcing steel have not been made, average properties of the various lots as determined to date are given in Table 5. It is planned to test coupons which will indicate the properties of the steel in each individual column.

The reinforcement used in Series 1, made before the main stock of steel was received, differs slightly from the remainder in the above list. The spirals of cold drawn wire were intended to approximate the quality to be obtained in a hot rolled rod of intermediate grade. The longitudinal steel was nearly the same as that received later and used in the remaining series.

3. *Details of Reinforcement*—The 8-in. columns were designed to have three percentages of longitudinal steel, 1.5, 4, and 6, and two percentages of spiral, 1 and 2. The larger columns were

TABLE 5—AVERAGE TENSILE PROPERTIES OF REINFORCING STEEL

Nominal sizes of bar used in all computations.

Bar Size in.	No. of Tests	Grade of Steel	Yield Point* lb. per sq. in.			Ultimate Strength lb. per sq. in.	Per Cent Elong. in 8 In.	Used in Series No.
			Max.	Min.	Av.			
			Longitudinal Steel					
½ rd.	24	Int.	50,600	43,800	45,600	73,300	19.5	2, 3
½ sq.	3	Int.	48,300	47,700	47,900	74,900	22.3	1
½ sq.	182	Int.	57,000	50,800	53,400	83,200	21.4	2, 3, 6, 7
½ sq.	26	Struct.	42,100	36,700	40,400	59,300	28.2	2
½ sq.	24	Rail	75,400	63,000	68,300	109,100	16.0	2
⅝ rd.	22	Int.	43,600	35,800	39,300	61,800	27.1	2, 3, 7
¾ rd.	32	Int.	54,000	48,600	51,100	84,500	21.2	2, 3, 7
⅞ rd.	45	Int.	43,300	40,200	41,700	69,700	26.0	5, 6
1 rd.	16	Int.	56,200	48,700	50,400	83,900	23.3	5, 6
1⅛ sq.	8	Int.	46,300	43,400	45,300	79,000	19.0	5, 6
			Hot Rolled Spiral Rod					
No. 5	19	Int.	53,000	44,500	49,400	79,500	14.5	2, 3, 6
¼	69	Int.	52,800	39,700	48,200	74,200	19.0	3, 5, 6
5/16	10	Int.	47,100	39,700	44,500	75,500	20.0	5, 6
⅜	25	Int.	45,200	38,000	41,300	71,000	22.6	5, 6
			Bright Cold Drawn Wire					
3/16	2	Wire	49,000	47,000	48,000	81,900	....	1
No. 5	6	Wire	85,200	75,300	80,400	90,000	3.0	7
¼	6	Wire	90,000	74,500	83,200	109,400	4.3	7

Rail steel bars were plain sq., all others deformed.

*No yield point on spiral stock. Values given represent a "useful limit" of the material, and are stresses at a unit elongation of 0.005. This elongation is based upon observations of spiral strains at the ultimate load on columns made with various percentages of spiral reinforcement. (See Bulletin 190, Univ. of Ill. Eng. Expt. Sta.)

designed for 1.5 or 4 per cent of longitudinal steel and 1 per cent of spiral. The commercial sizes chosen to meet these designed values are given in Table 6, to which reference may be made in connection with the tabulated results of all tests. The number of bars selected are in multiples of 4, to facilitate strain measurements on four sides of each column. It is seen that the largest amount of longitudinal steel used in the 28-in. columns was 3.28 per cent. The original design of 4 per cent was reduced in this case to obviate the chance of exceeding the capacity of the 3,000,000-lb. testing machine. Another variation from the design percentage is seen in the 1.24 per cent spirals for 8-in. columns. With the smallest size of hot-rolled spiral rod available this was the smallest percentage possible without exceeding a pitch of one-sixth of the core diameter.

TABLE 6—DETAILS OF REINFORCEMENT USED IN TEST COLUMNS

Nominal sizes of reinforcement are used in all calculations.

Core* Diam. in.	Height ft. - in.	Longitudinal Bars			Spirals			
		No. and Size	Per Cent	Used in Series No.	Size	Pitch in.	Per Cent	Used in Series No.
8	5 - 0	4 - ½″ rd. 8 - ½″ sq.	1.57 3.98	2, 3 1,2,3,6,7	3/16″ No. 5 (.207″)	1.35 1.35	1.00 1.24	1 2, 3, 6, 7
		4 - ⅝″ rd. 4 - ¾″ rd.	5.94	2, 3, 7	¼″	1.19	2.00	3, 7
12	7 - 6	4 - ¾″ rd. 8 - ⅞″ rd.	1.56 4.25	5, 6 5, 6	¼″	1.60	1.00	5, 6
20	12 - 6	8 - ⅞″ rd. 16 - 1″ rd.	1.53 4.00	5, 6 5, 6	5/16″	1.51	1.00	5, 6
28	17 - 6	12 - 1″ rd. 16 - 1⅛″ sq.	1.53 3.28	5, 6 5, 6	⅜″	1.56	1.00	5, 6

*Percentages of reinforcement based on this core diameter, out to out, of spiral.

4. *Making and Storage of Columns*—The concrete for all columns was mixed in a one-bag batch mixer, the usual mixing time being 4 minutes. For the 8-in. columns all aggregates were reduced to a room-dry condition previous to use. Cement, aggregates and water were proportioned and measured out by weight. The 8-in. columns were made from a single batch, with sufficient excess to permit the making of six companion 6 by 12-in. cylinders. The concrete was wheeled to the forms in wheelbarrows and puddled with the aid of a steel tamping rod and a light air-hammer used on the outside of the form. In pouring the larger columns, each batch was hauled to the form in a bottom-dump bucket mounted on a truck, lifted by a 10-ton crane, and discharged into the form. The largest columns required about 25 batches of concrete and weighed about 8 tons.

The 8 and 12-in. forms were made of steel pipe, split into quadrants and held together with circular clamps. The larger forms were standard steel column forms, loaned by the Kalman Steel Co. for the purpose.

The vertical bars used in all columns were milled to exact length, then wired to the spirals to form a unit or cage. The unit was placed on a machined base plate, the form assembled around it and plumbed. To insure that the ends of bars remained in a plane, a machined 6-in. pipe flange (for the 8-in. columns) was forced and held down against the upper ends of the bars, and the concrete placed in the column through the 6-in. opening in the flange. For the larger columns it was found satisfactory to assemble the reinforcing unit in a vertical position on the base plate before wiring, with the result that the weight of the bars held them in contact at the bottom. In placing the concrete in the columns, the concrete was brought up to about even with the tops of the reinforcing bars. After the concrete had stiffened, a thin layer of neat cement paste or 1:1 mortar was applied to the top and a machined bearing plate pressed down on this cap until it rested firmly on the milled ends of the bars. With this technique it was not difficult to bring the ends of bars into the plane of the cement cap.

Forms were removed from the 8-in. columns the day after moulding, gage lines were prepared by exposing the reinforcing steel and by setting small steel plugs in the concrete with quick-setting cement. Gage holes were drilled for strain measurements with a No. 54 drill. Initial strain readings were taken and the columns were placed in the moist room for the remainder of the 56-day moist storage period. The air of the moist room was kept saturated by humidifying sprays and the temperature was held at 70 deg. F. with an average weekly variation of about one degree. Due to the size and weight of the large columns, it was impossible to place them in the moist room. They were stored in the main crane bay of the laboratory, under burlap which was wet down generally three times a day. The temperature of the room during this curing period—June to September—ranged from about 70 to 90 deg. F. The temperature beneath the wet burlap was considerably lower and the average value did not vary greatly from 70 deg. F.

5. *Testing Methods*—All columns except those of Series 3 were tested to failure in one of four testing machines, a 3,000,000-lb. Southwark-Emery hydraulic machine, a 600,000-lb. Riehle, a

TABLE 7—PRECISION OF STRAIN GAGES

Gage	Gage Length in.	Dial Graduation in.	Mult. Ratio	Estimated Precision* Millionths of inch per inch
Whittemore	10	0.0001	1	6
Berry No. 2040	10	0.001	5	10
Berry No. 2040	8	0.001	5	12
Berry No. 2280	8	0.001	5	8
Diameter gage	8	0.001	5	15
Howard type	10	0.001	1	150
Howard type	50	0.001	1	20

*Based upon general working condition of instrument.

300,000-lb. Riehle, and a 300,000-lb. Olsen screw-power testing machine. A rate of loading was chosen for the hydraulic machine which was comparable with the speeds obtained with the screw-power machines.

Load was applied to the columns through a top spherical bearing block, and in some cases through both top and bottom blocks. With the exception of tests of Series 2, after a small zero load was applied, these blocks were wedged in a fixed position, but it is quite evident that after any considerable load is applied such a block is held against rotation by friction alone. The columns were thus loaded as "flat-ended" columns.

Strain measurements were made by the use of various types of gages. In the majority of the work, longitudinal strains were measured with a 10-in. Whittemore gage at gage lines on four sides of the column on concrete and on steel. The centers of these lines were located 10 in. from the top and bottom of the column, and at midheight. Lateral deformations were measured by use of a "diameter" gage which indicated changes on diametral gage lines. Other gages were a 10-in. Berry gage, 8-in. Berry gages with long legs which were required on the columns having 2 in. of fireproofing, a 50-in. direct reading gage which was used to measure volume changes during moist storage, and a 10-in. direct reading gage for measuring large strains beyond the range of the Whittemore and Berry gages. A summary of the constants of the gages used and their probable precision is given in Table 7.

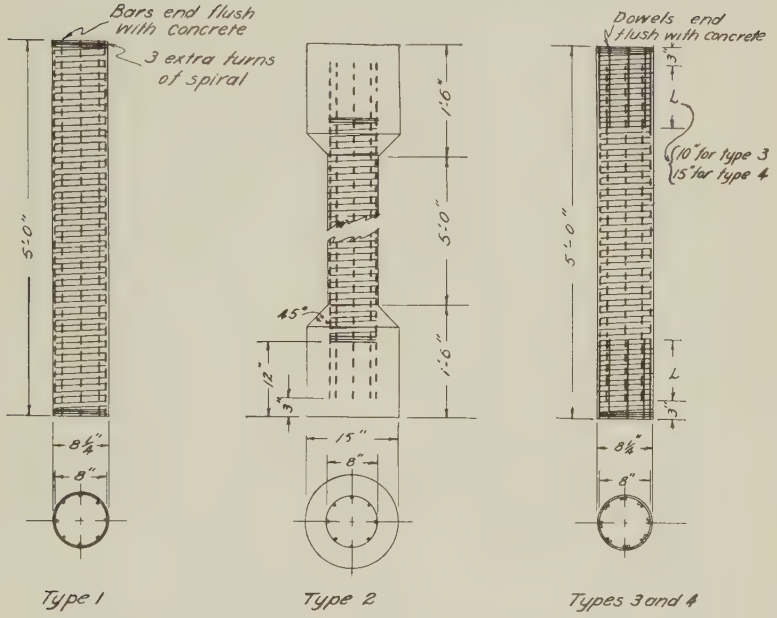


FIG. 1—TYPES OF COLUMN USED IN SERIES 1

SERIES 1—STUDY OF END CONDITIONS

6. *Results of Tests of Series 1*—This group of ten columns was designed to furnish information as to the most desirable type of end condition to be used in the columns of the later series of tests. Four types of end condition are included in the group, two columns of each type, except Type 1 for which there were four. Fig. 1 gives details of the four column types. All of the columns had eight ½-in. square bars (4 per cent) of intermediate grade, and $\frac{3}{16}$-in. cold drawn spirals, 8 in. outside diameter, and 1.35-in. pitch (1.0 per cent). The columns of Type 1 had the ends of reinforcing bars flush with the ends of the concrete. In Type 2, a capital 15 in. in diameter and 18 in. long was added at each end of the column shaft, making the total length 8 ft. The reinforcing bars extended into these capitals to a point 3 in. from the bearing faces, but the spirals were stopped 12 in. from the ends of the

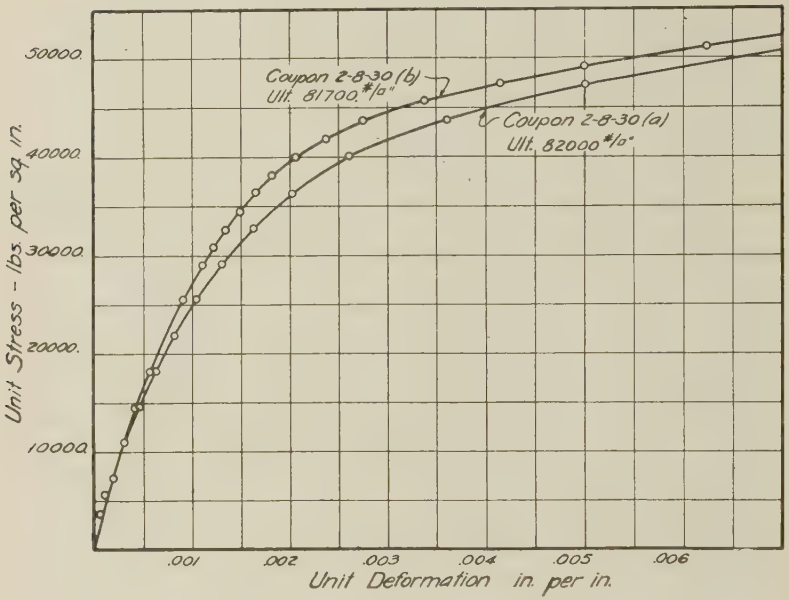


FIG. 2—STRESS-STRAIN CURVES FOR SPIRAL REINFORCEMENT, SERIES 1

column. No other reinforcement was used in the capitals. In Type 3 a 20-diameter splice was made in the longitudinal bars at each end of the column. This was made by stopping the main bars 3 in. from each column end and using 13-in. dowel bars which ended flush with the column ends. Type 4 was like Type 3 except that a 15-in. lap (30 diameters) was used. The dowels were wired to the main longitudinal bars.

Properties of the reinforcing steel are given in Table 5. The cold-drawn wire used in this series did not have a definite yield point; typical stress-strain curves are given in Fig. 2.

The concrete for these columns was intended to have a compressive strength of 3500 lb. per sq. in. at 28 days, but due mainly to the fact that no trial mixtures had been made with the cement used, the concrete obtained gave an average cylinder strength of 5195 lb. per sq. in. The concrete was made with different aggregates from those used in later tests, a torpedo sand from

Attica, Indiana, having a fineness modulus of 3.03 and a ¾-in. Attica gravel having a fineness modulus of 7.21. With the water-cement ratio used, 0.90, slumps of 2.8 to 6.3 in. were obtained. The proportions used, by weight, were 1:2.5:2.8.

The moulds were made from 8-in. steel pipe, cut into quadrants. To accommodate the three spacing bars on the spirals the mould diameter was increased slightly by the introduction of ¼-in. fillers between segments. Leakage of water from the concrete of these specimens led to the practice in later series of tests of sealing all joints with paraffin previous to casting of columns.

Following a procedure outlined by Committee 105, after the forms were stripped, the columns were given two coats of linseed oil before they were placed in moist storage. This was intended to prevent loss of moisture during temporary exposure to the air while gage lines were being prepared and also at the time of testing. However, in the later series this procedure was omitted as unessential and of doubtful value.

The columns were tested when 28 days old, in the 600,000-lb. Riehle testing machine. A spherical bearing block was used at top and bottom of all columns except the two having enlarged ends, with which only the upper bearing block was used. After columns were plumbed and a small initial or "zero" load was applied, the spherical blocks were wedged against further movement. After the initial load of about 6000 lb., a full set of strain readings was taken. The remaining load was applied in about 10 equal increments, and a complete set of strain readings was taken at each increment.

As previously noted, the strain readings were taken at three levels, at middle and 10 in. from each end of the column. In the columns with lap splices in the reinforcement, readings were taken only on the main bars; none on the short dowel bars. Fig. 3 shows views of columns of Types 1 and 2 during testing.

Since the spiral wire was of relatively low strength, approximating intermediate grade hot rolled rod, and since the concrete was of relatively high strength, the one per cent of spiral did not add a very large margin of strength beyond the yield point of any of the columns of Series 1. Little lateral deformation was

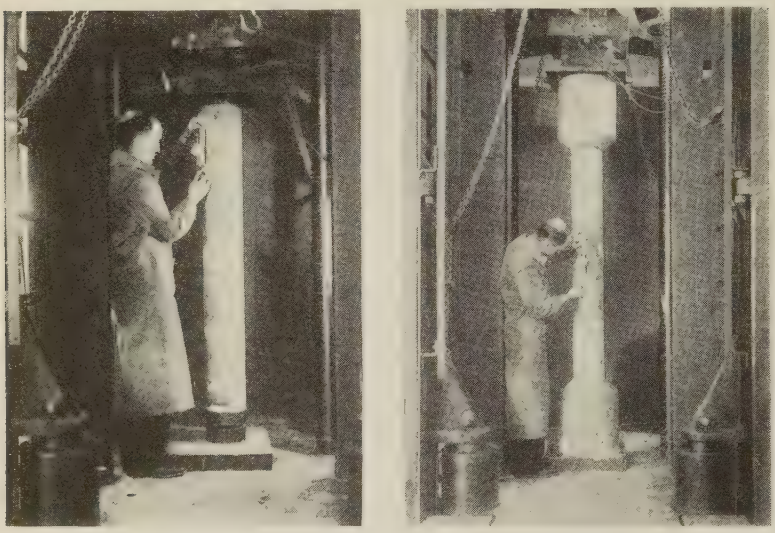


FIG. 3—COLUMNS OF TYPES 1 AND 2 DURING TEST, SERIES 1

noted until the yield point of the column was reached and no appreciable spalling occurred until shortly before the ultimate load was applied. The spiral wire did not break until the maximum load had been passed; then failure occurred by the snapping of one or more wires and outward buckling of the longitudinal bars.

The manner of failure of the two columns of Type 2, with capitals, is noteworthy. Column *g* failed in the shaft in much the same manner as the other columns but breakage of spiral wire after failure was forestalled by splitting of the capital at the top of the column. The strength of this column was equal to that of the strongest column of Type 1. Column *j* failed through the splitting of both capitals, apparently before the full strength of the shaft had been developed.

The principal results of the tests of Series 1 are given in Table 8, together with data on the quality of the concrete used. Both columns and companion cylinders showed unusual uniformity of strength. It is surprising that there was only 5 per cent difference

TABLE 8—RESULTS OF TESTS OF SERIES 1

Reinforcement 8½ in. squares, intermediate grade; Spirals, $\frac{3}{16}$-in. cold drawn wire, 1.35 in. pitch. Spirals 8 in. outside diameter, Core area 50.2 sq. in. Cylinder strengths represent average of three tests. Age of columns and cylinders at test, 28 days.

Type	Col. No.	End Conditions	Maximum Load			6 by 12-in. Cyl.-	
			lb.	lb. per sq. in.	Per Cent of Type 1	Slump in.	Comp. Strength lb. per sq. in.
1	a	Bars milled, ends flush with concrete	376,300	7500		6.3	5080
	b		367,000	7300		6.3	5080
	h		358,700	7150		4.2	5220
	k		374,100	7450		4.4	5310
	Av.		369,200	7350	100.0	5.3	5170
2	g	Enlarged capitals at ends	376,900	7510		4.4	5220
	j		347,000	6910		4.2	5310
	Av.		361,900	7210	98.1	4.3	5265
3	c	20-diam. lap of bars. Dowels milled and flush at ends	331,000	6600		2.8	5240
	d		375,300	7480		2.8	5240
	Av.		353,200	7040	95.6	2.8	5240
4	e	30-diam. lap of bars. Dowels milled and flush at ends	365,300	7280		6.3	5110
	f		345,300	6880		6.3	5110
	Av.		355,300	7080	96.2	6.3	5110

in average strength between the columns of Type 1, which were the highest, and those of Type 3, which were the lowest of the group. It will also be noted that the average compressive strength for all of the companion cylinders was 5195 lb. per sq. in. and that the variations in the concrete strengths for the four different types of columns did not exceed 1.6 per cent from this average value. Considering that the concrete furnished roughly 60 per cent of the strength of the column, it will be seen that the variations in concrete quality should not produce a difference in the strength of the different types of column amounting to more than 1 per cent. Accordingly no attempt has been made to correct the relative strengths of columns to allow for these minor variations in quality of concrete.

Various attempts have been made to account for the portion of the strength of a column contributed by each of its elements, and most column formulas indicate the share of the load carried on concrete, on vertical reinforcement and on spiral reinforcement. It is evident that the results of Series 1, comprising only 10 column tests and in which variable end conditions were introduced, should not be used to establish any general column formula. The object of the following tabulation is merely to present the results in a form that may easily be studied when the results of the later and more comprehensive series of tests are available. A well-established method of expressing the strength of a column is to consider it made up of three elements, (1) the ultimate strength of the net concrete core area, considered as plain concrete, (2) the load required to stress the compressive reinforcement to the yield point, and (3) the load required to stress the spiral to its limit of effectiveness. It is definitely known that the strength of the concrete in a column seven and one-half diameters in length should be less than that of a 6 by 12-in. cylinder. Tests in this laboratory in 1924 and 1925 showed the strength of 8 by 40-in. and 10 by 40-in. plain columns to be about 85 per cent of that of companion cylinders; the results of Series 2 for 8 by 60-in. plain column (see Table 10, to follow) show a column-cylinder strength ratio of about 0.80. The corresponding tests of Series 2 made at Lehigh University show a ratio of about 0.85 and in the present studies this value will be used. Since no plain columns were included in Series 1, an estimate of the load carried by the concrete portion of the column will be taken at 85 per cent of the companion cylinder strength. The longitudinal steel consisted of 3.98 per cent of intermediate grade, having a yield point of 47,900 lb. per sq. in. There is no definite yield point of the spiral steel; however, an estimate of the useful limit of strength of spiral may be made on the basis of previous tests,[1] in which the spiral strain at the ultimate column load was found to vary from 0.003 to 0.009. Accordingly, the stress in the spiral wire at a strain of 0.005 has been chosen as a rough measure of the useful limit of the spiral reinforcement; the average value found being 48,000 lb. per sq. in. Table 9 gives the distribution

[1]Bulletin 190, Engineering Experiment Station, Univ. of Illinois.

TABLE 9—ANALYSIS OF DISTRIBUTION OF COLUMN STRENGTH, SERIES 1

Type of Col.	End Condition	Column Strength P/A lb. per sq. in.	Cylinder Strength f'_c lb. per sq. in.	Portion Col. Str. Attributed to		
				Concrete*	Vert. Steel	Spiral Steel
				$0.85f'_c$ x 0.96 lb. per sq. in.	(3.98 per cent $f_s = 47900$) lb. per sq. in.	(1.0 per cent $f_s = 48000$) lb. per sq. in.
1	Ends flush	7350	5170	4215	1900	1235
2	Capitals	7210	5265	4290	1900	1020
3	20-d lap	7040	5240	4270	1900	870
4	30-d	7080	5110	4165	1900	1015
Average		7170	5195	4235	1900	1035

*Concrete section is 96 per cent of core area.

of the strength of the columns of Series 1 on the basis of the above method of calculation. It must be remembered that this calculation has thrown most of the variability of the column strengths into that portion of the strength attributed to the spiral reinforcement, whereas it is much more probable that such variations were due to the different arrangements of the vertical steel.

The longitudinal strains measured on all columns of the series are shown on the load-strain curves of Fig. 4. The strains in concrete and steel at the different elevations are seen to be similar, except for the columns of Types 3 and 4, in which dowels were used. In these columns, the strains measured near the ends of the main reinforcing bars are generally less than those in the adjoining concrete. This is because the gage line is within the lap splice in the reinforcement.

The variation between strains at top, middle and bottom of each column may be seen in Fig. 4. While the point at which high early strains developed usually coincided with the point of failure, no consistent rule may be stated for the location of the point of failure in the columns. In those with dowels, failure occurred within the zone of the reinforcement splice, and in the others the greater number of failures were in the upper third of the column shaft.

The modulus of elasticity of the concrete used in the columns, as found from tests of 6 by 12-in. cylinders, averaged 4,320,000 lb. per sq. in. for the 10 columns. The lower portion of the stress-

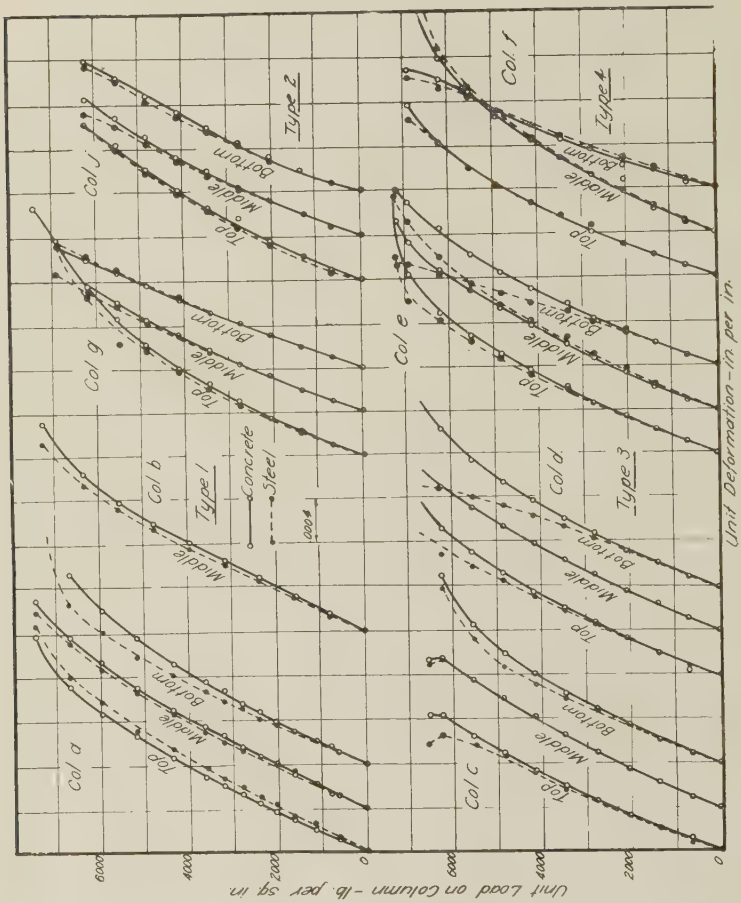


FIG. 4—STRESS-STRAIN CURVES FOR COLUMNS OF SERIES 1

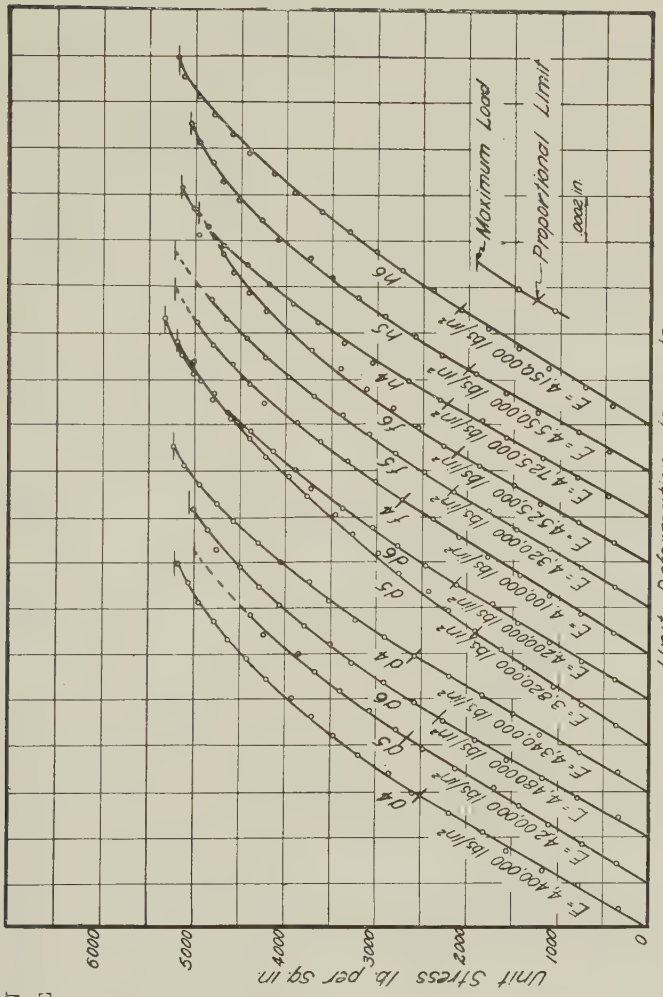


FIG. 5—STRESS-STRAIN CURVES FOR CONCRETE OF SERIES 1

strain curve for these cylinders was a straight line. The limit of proportionality for the concrete was found to be 37 to 54 per cent of the ultimate strength, with an average of 44 per cent. Fig. 5 shows typical stress-strain diagrams for this concrete.

From a consideration of the relative strengths of the columns of the four types given in Table 8, it might be concluded that any one of the four types would give satisfactory results with the possible exception of the type having enlarged ends. However, since there was no apparent advantage with either the type having capitals or those having dowels, economy of material and labor pointed to the selection of Type 1. This type was used in all of the remaining columns of the investigation with the exception of a few columns of Series 8 which were designed to show the effect of long-continued loading on the transfer of stress to the longitudinal reinforcement by bond alone. It might be noted that the amount of material required for the type with capitals was roughly twice as great as for the one finally chosen, and that the labor of making and testing such columns and the moist storage room required for them would be much more than twice as great. The use of dowel bars showed no particular advantage or increase in information to be secured, and an objection to their use was found in the difficulty of securing proper contact of the milled ends against the bearing plates. The wisdom of the choice of Type 1 for the succeeding series of tests has been borne out in the conduct of the later series.

SERIES 2—STUDY OF METHOD OF LOADING

7. *Results of Tests of Series 2*—This series of tests was planned to give information on a number of questions relating to column strength, including different methods of loading, a considerable range in kind and amount of reinforcement and in quality of concrete. The columns were made in general with three grades of concrete, designed to produce 56-day strengths of 2000, 3500 and 5000 lb. per sq. in. In addition a small group was made with concrete designed to produce a 56-day strength of 8000 lb. per sq. in., when compacted into place by the use of an electric vibrator.

Longitudinal reinforcement of three different grades was included in the series and in general three percentages ranging from

1½ to 6 per cent were used. The grades of steel used were specified to meet requirements for intermediate, structural, and rail steel. The spiral reinforcing used was of intermediate grade, hot rolled rod; about 1¼ per cent was used in all columns of this series.

As noted, one of the principal studies embodied in the series was that of rate of loading of the test column. The method used in practically all previous column tests has been to apply the load in increments, allowing only sufficient time between increments for the observation of strains. It was felt that in the range of plastic action of the column a longer period of time between increments would produce a considerable amount of plastic deformation and might quite possibly have a decided effect upon the carrying capacity of the column. Accordingly, two methods of testing were adopted. These will be denoted in the following discussion as "fast" and "slow" loading. Under the "fast" loading the load is applied to the column in 10 or more increments with a time interval between increments of 15 minutes or less, during which strain measurements were taken. In the "slow" loading, a load of approximately ¾ of the estimated yield point strength of the column was applied in 8 to 10 equal increments at the same rate of loading as used in the fast loading method. Subsequent loads were applied in 6 to 10 increments at intervals of four hours. With this procedure, strain readings were taken during each 4-hr. interval immediately before and after the application of an increment of load.

The amount of flow in the test piece during a 4-hr. period naturally caused a certain amount of release of load in the testing machine. To minimize this condition for the tests under slow loading, a special load-holding device was used. This consisted of a bed of coiled springs having a total capacity equal to or greater than the column strength and having a total shortening of approximately 1½ in. The shortening of a column due to plastic yielding of a few thousandths of an inch produced very little difference in the pressure exerted by this bed of springs and hence the load was held nearly constant throughout a full 4-hr. period. A view of a column as tested with one of these devices is shown in Fig. 6. Another feature of the tests with slow loading, which lasted from 36 to 48 hours, was the use of a protective case

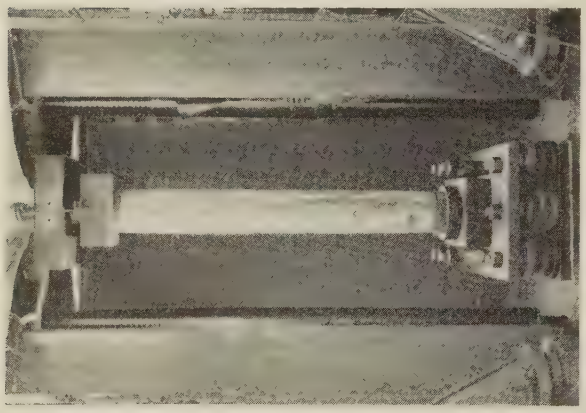


FIG. 7—PROTECTIVE CASE USED WITH SLOW LOADING TESTS, SERIES 2

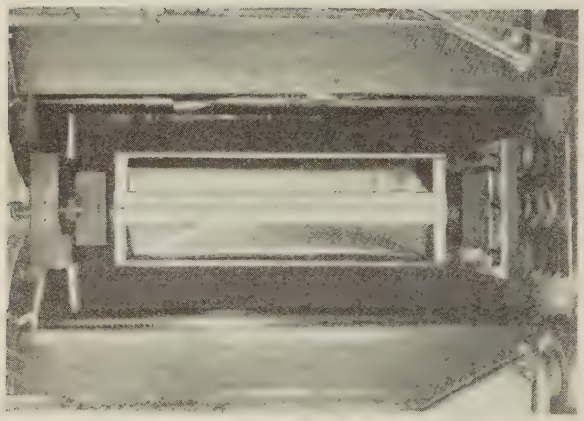


FIG. 6—COLUMN ON TESTING BASE, SLOW LOADING, SERIES 2

which was placed around the column to prevent its drying out. Such a case, lined with wet burlap, is seen in the view of Fig. 7.

The tabulated results of all of the tests of Series 2 are given in Table 10. In all but the tests in which 8000-lb. concrete was used, there were companion specimens tested under both the fast and slow loading, two columns of a kind. In addition to the columns scheduled in the 1930 committee report previously mentioned, 27 plain columns were also made and tested to give further information on the action of the concrete alone under fast and slow loading.

The data of Table 10 indicate that there was not a very marked difference in the carrying capacity of columns tested under the two methods of loading. There was, however, a considerable difference in the amount of deformation measured as indicated by the typical stress-strain curves for corresponding gage lines of columns tested under fast and slow loading, as shown in Fig. 8. From these results it appears that the amount of flow which takes place during a 4-hr. period, while producing quite appreciable changes in length of the test piece, has not produced appreciable changes in strength, except perhaps for the plain columns. The difference in action between plain and spirally reinforced columns is quite readily explained. As has been found in previous investigations, after plain concrete has reached 75 to 85 per cent of its ultimate strength, a decided change in its action takes place wherein lateral bulging develops, internal breakdown of the structure begins, and plastic deformation increases rapidly. The failure of plain concrete may then be expected to take place sooner under the slow loading than under the fast loading. When concrete is restrained within an envelope of spiral reinforcement, it soon reaches a stage of almost entirely plastic deformation wherein the loads carried are in a definite proportion to the lateral pressure exerted by the spiral reinforcement. This proportion is independent of the amount of lateral deformation and hence the column may be expected to carry a given load as long as a definite pressure is exerted by the spiral reinforcement. Failure evidently occurs when the spiral becomes unable to furnish increased lateral pressure with an increase in lateral deformation. Whether it would ever be possible to have sufficient plastic de-

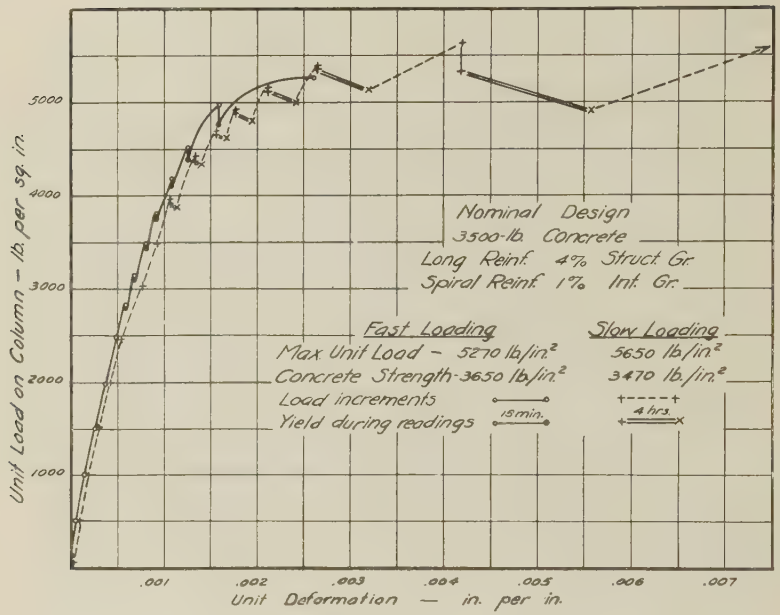


FIG. 8—STRESS-STRAIN CURVES FOR FAST AND SLOW LOADING, SERIES 2

formation or flow of the concrete between spiral wires to cause a decrease in the pressure exerted by the spiral reinforcement is problematic. The relation between the deformations of columns under fast and slow loading will be discussed further in a later report.

A comparison of ultimate column strengths obtained under fast and slow loading methods may be made by reference to Fig. 9, in which column strengths are plotted against the strength of concrete control cylinders. The figure contains the results from all columns of the series made with 1.24 per cent of spiral reinforcement and 3.98 per cent of longitudinal reinforcement of the three grades shown as well as the strength of the plain concrete columns. It is seen that there was no consistent difference in the strengths obtained by the two methods of testing; such differences as are found might be considered as within the range of variation

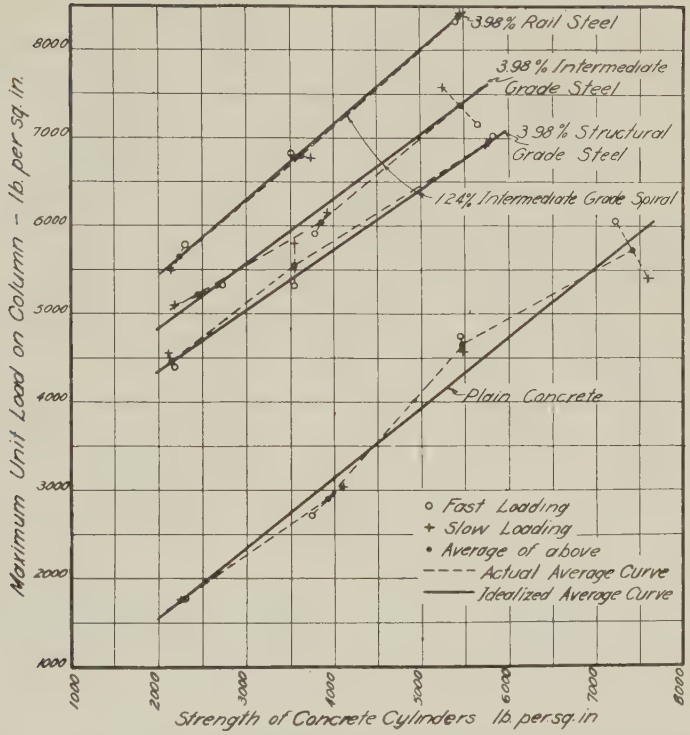


FIG. 9—RELATION BETWEEN COLUMN STRENGTH AND STRENGTH OF CONTROL CYLINDERS, FAST AND SLOW LOADING, SERIES 2

to be met had a single method of loading been used. For this reason, curves have been drawn to represent the average results from the two methods of loading. In plotting these curves the strengths of two individual columns, noted in Table 10 as having shown excessive deflection before failure, have been neglected. One of these columns was thrown clear out of the testing machine at failure, and both appear much weaker than their companion columns.

The curves of Fig. 9 also permit a study of the effect of the three grades of longitudinal reinforcement, and of the three concrete mixtures employed. The points for columns made with rail

steel fall upon a straight line which is practically parallel to the average curve for plain concrete columns in the lower part of the diagram. For columns made with bars of intermediate and structural steel, the plotted points are represented in two ways; the average points are connected by light dotted lines, which represent actual test results; and in addition, heavy solid lines are drawn which fit the respective sets of points reasonably well. In drawing these latter curves, less weight has been given to the points representing 3500-lb. concrete than to the others. It will be noted that the slopes of these two curves are slightly less than the one for plain concrete columns. The average slope for the four curves shown is slightly less than 0.8. This verifies previous tests which indicate that the difference in strength of spirally reinforced columns due to differences in concrete mixtures is equal to the difference in strength of plain columns of these mixtures, times the quantity $(1-p)$. That is, the slopes of the upper three curves might be expected to be 96 per cent of that for plain columns.

The intersections of the solid average curves of Fig. 9 with verticals representing concrete strengths of 2000, 3500 and 5000 lb. per sq. in. have been plotted in Fig. 10, correction being made in this way for variations in actual cylinder strengths from the designed values. Fig. 10 shows column strengths plotted against the average yield point stress for each grade of longitudinal reinforcement. Such a comparison is admittedly inaccurate since there was a considerable variation in the yield point stress for each grade of steel, but corrections for variations in the reinforcement in individual columns cannot be made until the tests of steel coupons are completed. (Test coupons have been saved from the two 22-ft. lengths of bar used in each column of this group.) Fig. 10 shows, however, that the increase in column strength is nearly proportional to the increase in average yield point stress for the reinforcement. With 3.98 per cent of reinforcement, an increase in yield point stress from 40,400 to 68,300 lb. per sq. in. should produce an increase in column strength of 0.0398 times 27,900 or 1110 lb. per sq. in., while the average increase shown for the three curves is 1300 lb. per sq. in. The rate of increase is nearly the same for the columns made with the three different grades of concrete.

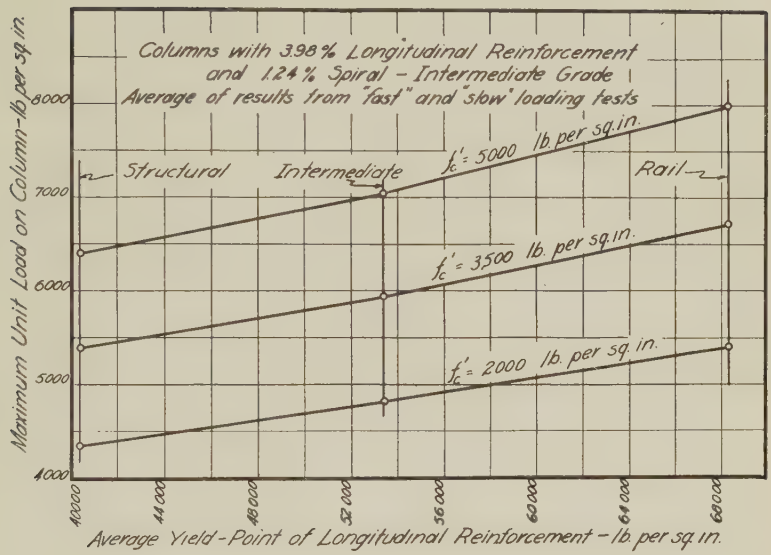


FIG. 10—RELATION BETWEEN COLUMN STRENGTH AND YIELD POINT STRESS FOR LONGITUDINAL REINFORCEMENT, SERIES 2

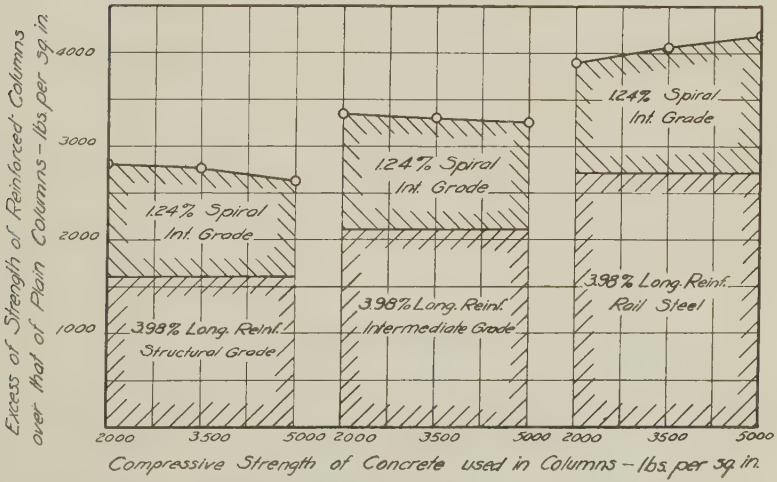


FIG. 11—CALCULATED DISTRIBUTION OF COLUMN STRENGTH BETWEEN LONGITUDINAL AND SPIRAL REINFORCEMENT, SERIES 2

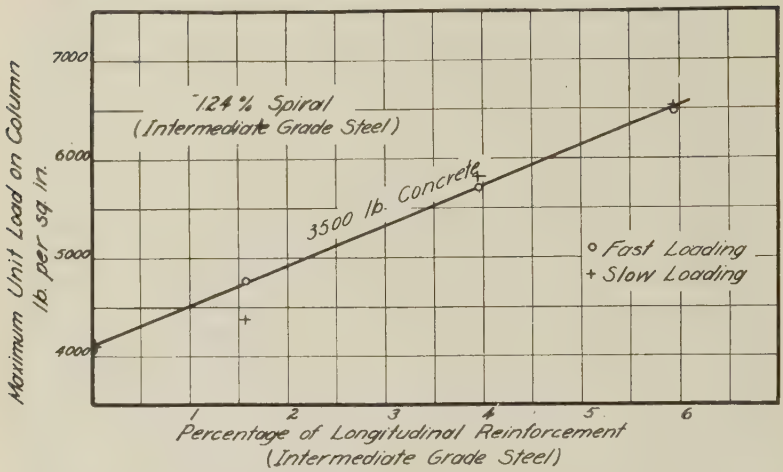


FIG. 12—RELATION BETWEEN COLUMN STRENGTH AND PERCENTAGE OF LONGITUDINAL REINFORCEMENT, SERIES 2

In studying the distribution of column load between the concrete section, the longitudinal steel and the spiral steel, further reference may be made to Fig. 9. Considering that the concrete of the reinforced columns carried a stress equal to the strength of the plain columns on a section 96 per cent of the core area, the portion of the column strength to be attributed to the reinforcement is found, and is shown by the plotted points in Fig. 11. This is distributed between longitudinal and spiral reinforcement, by computing the load on the former, 0.0398 times the yield point stress. The shaded areas indicate the relative portions of the column load attributed to each form of reinforcement. In view of the fact that accidental variations in column strength have thus been thrown into the portion attributed to the spiral reinforcement it is of interest that this portion is so nearly constant for the nine combinations of concrete and longitudinal steel shown.

Another group of columns of this series was made with 3500-lb. concrete, 1.24 per cent of intermediate grade spiral, and 0, 1.57, 3.98 and 5.94 per cent of intermediate grade longitudinal steel. The column strengths found with both fast and slow loading are

plotted in Fig. 12 against percentages of longitudinal reinforcement. Variations in concrete strength as shown by the control cylinders for these columns have been corrected by the use of the average curve for plain columns given in Fig. 9, and the column strengths shown in Fig. 12 correspond to cylinder strengths of 3500 lb. per sq. in. As in Fig. 9, there is little difference in the results for fast and slow loading. The test values are represented by an average curve for the two loadings that is nearly a straight line. Since there is some variation in the quality of the steel used to produce the three percentages, the average curve should not be exactly linear. The increase in strength with increase in longitudinal reinforcement is slightly less than that computed from the percentages and the average yield point stresses as listed in Table 5. The average yield point for the 1.57 per cent steel was 45,600 lb. per sq. in.; that for 3.98 per cent, 53,400 lb. per sq. in.; and for 5.94 per cent, 45,200 lb. per sq. in. On this basis the contribution of the three percentages of longitudinal reinforcement to the column strength should be 715, 2150 and 2685 lb. per sq. in. respectively. From Fig. 9, 3500-lb. concrete should contribute to the column strength about 2750 (1-p) lb. per sq. in. This leaves a remainder which may be attributed to the 1.24 per cent of spiral reinforcement averaging 1165 lb. per sq. in. for the four sets of test values.

Series 2 also included tests of columns made of concrete designed to give a 56-day cylinder strength of 8000 lb. per sq. in. The tests were made by the fast loading method only. The strengths obtained with this rich concrete were quite variable, and since a shortage of reinforcing spirals delayed the testing of part of the group until January, 1931, no analysis of the test results will be given here. The principal results for the group are given in Table 10.

Further analyses of the results of Series 1 and 2, on the basis of more complete information regarding the reinforcement, and in the light of the results of the remaining parts of the investigation, will be made in a later report.

TABLE 10—RESULTS OF TESTS OF SERIES 2

All reinforced columns 8 in. core diameter, 5 ft. long, 2 companion columns of each type Columns cured 56 days in moist room and tested moist. All spiral reinforcement of intermediate grade steel. Plain columns 8¼-in. diameter.

Col. No.	Designed Concrete Str. lb. per sq. in.	Percentage of Reinforcement		Column Str. Fast Loading		Cyl. Str. lb. per sq. in.	Column Str. Slow Loading		Cyl. Str. lb. per sq. in.
		Long.	Spiral	Total lb.	Unit lb. per sq. in.		Total lb.	Unit lb. per sq. in.	
Intermediate Grade Vertical Bars									
a	2000	3.98	1.24	269,000	5350	2950	252,000	5000	2180
b	2000	3.98	1.24	267,500	5310	2490	240,000	4770	2100
c							262,800	5210	2230
d							269,000	5340	2135
Average				268,250	5330	2720	255,950	5080	2160
a	3500	3.98	1.24	276,000	5490*	3530	310,000	6150	3910
b	3500	3.98	1.24	298,000	5920	3770	271,000	5380*	3790
Average				287,000	5705	3650	290,500	5765	3850
a	5000	3.98	1.24	362000,	7200	5550	383,000	7600	4880
b	5000	3.98	1.24	360,000	7160	5710	381,000	7560	5530
Average				361,000	7180	5630	382,000	7580	5205
Structural Grade Vertical Bars									
a	2000	3.98	1.24	217,000	4310	2310	231,000	4590	2150
b	2000	3.98	1.24	226,000	4490	2050	227,000	4510	2060
Average				221,500	4400	2180	229,000	4550	2105
a	3500	3.98	1.24	265,000	5270	3650	284,000	5650	3470
b	3500	3.98	1.24	271,000	5390	3450	300,000	5950	3590
Average				268,000	5330	3550	292,000	5800	3530
a	5000	3.98	1.24	352,000	7000	6080	360,000	7150	5640
b	5000	3.98	1.24	354,000	7040	5620	336,000	6670	5800
Average				353,000	7020	5850	348,000	6910	5720
Rail Steel Vertical Bars									
a	2000	3.98	1.24	288,000	5730	2250	267,000	5300	2070
b	2000	3.98	1.24	294,000	5850	2330	285,500	5680	2170
Average				291,000	5790	2290	276,250	5490	2120
a	3500	3.98	1.24	340,000	6760	3550	350,000	6950	3770
b	3500	3.98	1.24	347,800	6900	3450	330,200	6570	3570
Average				343,900	6830	3500	340,100	6760	3670
a	5000	3.98	1.24	456,000	9080	5510	414,500	8240	5500
b	5000	3.98	1.24	378,500	7540	5200	433,800	8600	5350
Average				417,250	8310	5355	424,150	8420	5425

*These columns showed large deflections at failure.

TABLE 10—RESULTS OF TESTS OF SERIES 2—CONTINUED

All reinforced columns 8 in. core diameter, 5 ft. long, 2 companion columns of each type. Columns cured 56 days in moist room and tested moist. All spiral reinforcement of intermediate grade steel. Plain columns 8¼-in. diameter.

Col. No.	Designed Concrete Str. lb. per sq. in.	Percentage of Reinforcement		Column Str. Fast Loading		Cyl. Str. lb. per sq. in.	Column Str. Slow Loading		Cyl. Str. lb. per sq. in.
		Long.	Spiral	Total lb.	Unit lb. per sq. in.		Total lb.	Unit lb. per sq. in.	
Intermediate Grade Vertical Bars									
a	3500	0	1.24	209,000	4150	3560	201,500	4000	3580
b	3500	0	1.24	208,000	4130	3660	212,600	4220	3490
Average				208,500	4140	3610	207,050	4110	3535
a	3500	1.57	1.24	226,000	4500	3440	234,000	4650	3870
b	3500	1.57	1.24	241,500	4800	3290	243,500	4840	4050
Average				233,700	4650	3365	238,750	4745	3960
a	3500	5.94	1.24	314,000	6240	3660	323,000	6420	3190
b	3500	5.94	1.24	337,000	6700	3280	327,200	6500	3630
Average				325,500	6470	3470	325,100	6460	3410
a	8000	0	1.24	390,000	7760	6920			
b	8000	0	1.24	421,000	8380	6960			
Average				405,500	8070	6940			
a	8000	1.57	1.24	400,000	7950	7570			
b	8000	1.57	1.24	420,200	8350	7310			
Average				410,100	8150	7440			
a	8000	3.98	1.24	463,000	9200	7690			
b	8000	3.98	1.24	482,000	9600	6790			
Average				472,500	9400	7240			
a	8000	5.94	1.24	488,000	9710	7310			
b	8000	5.94	1.24	450,000	8950	7100			
Average				469,000	9330	7205			

Table 10 concluded next page.

TABLE 10—RESULTS OF TESTS OF SERIES 2—CONTINUED

All reinforced columns 8 in. core diameter, 5 ft. long, 2 companion columns of each type. Columns cured 56 days in moist room and tested moist. All spiral reinforcement of intermediate grade steel. Plain columns 8¼-in. diameter.

Col. No.	Designed Concrete Str. lb. per sq. in.	Percentage of Reinforcement		Column Str. Fast Loading		Cyl. Str. lb. per sq. in.	Column Str. Slow Loading		Cyl. Str. lb. per sq. in.
		Long.	Spiral	Total lb.	Unit lb. per sq. in.		Total lb.	Unit lb. per sq. in.	
Plain Concrete									
a	2000	0	0	89,000	1665	2170	85,000	1590	2280
b	2000			100,000	1870	2125			
c	2000			100,000	1870	2505	103,000	1930	2240
d	2000			89,500	1675	2400			
Average				94,600	1770	2300	94,000	1760	2260
a	3500	0	0	140,000	2620	4100			
e	3500			145,000	2720	3570	Broken		
f	3500			139,000	2600	3570	162,400	3040	4100
c	3500			160,000	3000	3740			
Average				146,000	2730	3745			
b	5000	0	0	260,300	4870	5850			
c	5000			246,000	4600	5010	253,600	4740	5520
e	5000			268,100	5020	5435			
d	5000			240,000	4500	....	236,000	4420	5450
Average				253,600	4760	5430	244,800	4580	5485
a	8000	0	0	333,000	6230	6820			
b	8000			336,000	6290	6840	302,000	5650	8165
f	8000			282,500	5470	7480			
g	8000			335,000	6260	7720	276,000	5160	7015
Average				324,100	6060	7215	289,000	5400	7590

Acknowledgment—The investigation has required the help of a considerable laboratory staff. Especial acknowledgment is made to R. L. Brown, Research Associate, and to L. R. Tucker and T. G. Taylor, Research Graduate Assistants, who have given valuable assistance in the testing program, and in the reduction of the data.

Second Progress Report on Column Tests at the University of Illinois

BY F. E. RICHART* AND G. C. STAEHLE†

SERIES 3—COLUMNS UNDER LONG TIME LOADS

1. *Introduction.*—This report is a continuation of the progress report given in the A. C. I. Journal, February, 1931[1], on the results of the Reinforced Concrete Column Investigation. The previous report described the part of the investigation being conducted at the University of Illinois in cooperation with the American Concrete Institute and Lehigh University, gave details of the methods used in making and testing columns, and included a preliminary report on the tests of Series 1, "A Study of End Conditions of Column," and Series 2, "A Study of Method of Loading."[1] The outline of the entire investigation, prepared by Committee 105 of the Institute is given in the 1930 report of Committee 105[2].

The tests at the University of Illinois have been carried on as a cooperative research project of the Engineering Experiment Station, under the administrative direction of Dean M. S. Ketchum, Director of the Station, and Professor M. L. Enger, Head of the Department of Theoretical and Applied Mechanics. Acknowledgment is made of the assistance given by L. R. Tucker and T. G. Taylor, members of the Experiment Station Staff, in the testing work and the reduction of the data.

Due to the lack of time for consideration of the test results by the other members of Committee 105, this report (like the previous one) has been prepared as an individual report by those

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[1]Progress Report on Column Tests at the University of Illinois, *Proc. A. C. I.*, V. 27, p. 731, 1931.

[2]Progress Report of Committee 105, *Proc. A. C. I.*, V. 26, p. 601, 1930.

in charge of the tests at the University. However, it is desired to give full acknowledgment of the help of all members of the Committee, who participated in planning the tests, furnished many facilities for the work and advised concerning all phases of the investigation.

2. *Object and Outline of Tests of Series 3.*—It has been realized for many years that the shrinkage of concrete will set up stresses in a reinforced concrete column, and great emphasis was placed on this effect of shrinkage and the similar one of time yield or plastic flow by McMillan[3] in 1921. The purpose of the present series of tests was (1) to make a systematic study of the amount of shrinkage and plastic flow produced in reinforced concrete columns under the ordinary design load, continuously applied, and (2) having induced a redistribution of stresses in a column due to the action of shrinkage and flow over a period of a year, to note the effect upon the strength of the column as found by testing it to failure under the usual method of loading.

The series as outlined included 108 columns, of which 60 are to be held for a year under design load and 48 are duplicate columns held in similar storage but under no load. The design loads have been computed from the column formulas of the A. C. I. Joint Building Code, except for four columns, which have been loaded according to the design formula of the 1916 building code for New York City.

A list of the columns of Series 3, including the design loads used, is given in Table 1. All columns were made and cured in the moist room for 56 days. They were then placed in one of the following four conditions, of loading and storage, to be maintained for a period of one year:

(1) Under design load, in air of laboratory.
(2) Under no load, in air of laboratory.
(3) Under design load, in moist room.
(4) Under no load, in moist room.

The columns stored in air are subject to temperatures of 70 to 90 deg. F., and to relative humidities of 40 to 90 per cent. Those in the moist room are at temperatures of 70 deg. ± 4 deg., in a saturated atmosphere. Systematic strain measurements are

[3]A Study of Column Test Data, by F. R. McMillan, Proc. A. C. I., V. 17, p. 150-181, 1921

TABLE 1. DATA OF COLUMNS OF SERIES 3

Two companion columns of each type, except plain columns. All columns have 8-in. core diameter, 8¼ in. overall diameter, and are 5 ft. long. All columns cured 56 days in moist room, preceding the one-year period of loading and strain measurement. Design loads based on A. C. I. Joint Building code formulas, except for 4 columns indicated by *, which were loaded according to 1916 New York Code formula. All reinforcement is of intermediate grade steel. Columns marked † were loaded in 1931, all others in 1930.

Design Concrete Strength Lb. per sq. in.	Percentage of Reinforcement		Design Load on Column Lb.	Columns Loaded			Columns not Loaded	
	Vertical	Spiral		No.	Cylinder Strength Lb. per sq. in.	Date	No.	Cylinder Strength Lb. per sq. in.
			Air Storage					
2000	1.57	2.00	38,000	2	2220	8–21	2	2115
	3.98	0	37,200	2	2375	6–11	2	2535
	3.98	1.24	64,000	2	2245	7–31	2	2280
	3.98	2.00	64,000	2	2520	7– 7	2	2740
	5.94	2.00	90,000	2	2175	8– 7	2	2210
3500	1.57	0	47,200	2	3850	5–28	..	....
	1.57	1.24	52,400	2	3405	8–11	..	....
	1.57	2.00	52,400	2	3475	8–12	2	3770
	1.57	2.00	67,000*	2	3555	8–14	..	....
	3.98	0	55,200	2	4110	6– 4	2	4280
	3.98	1.24	79,000	2	3700	7–17	2	3795
	3.98	2.00	79,000	2	4240	6–25	2	3965
	5.94	0	81,200	2	3975	6–19	..	....
	5.94	1.24	108,600	2	3620	8– 4	..	....
	5.94	2.00	108,600	2	3520	8–15	2	4125
	5.94	2.00	75,200*	2	3545	8–26	..	....
5000	1.57	2.00	59,400	2	5570	8–19	2	5515
	3.98	0	72,000	2	5230	6– 9	2	5600
	3.98	1.24	96,000	2	5125	7–23	2	5215
	3.98	2.00	96,000	2	6105	7– 2	2	5860
	5.94	2.00	130,400	2	5690	8– 5	2	5435
2000	0	0	26,700	2	....	2–12†	2	....
3500	0	0	46,600	2	....	1–29†	2	....
5000	0	0	66,800	2	....	3– 2†	2	....
2000	0	0	0	..	....	Sept.	6	2420
3500	0	0	0	..	....	Sept.	7	4015
5000	0	0	0	..	....	Sept.	6	5185
			Moist Storage					
2000	3.98	0	37,200	2	2640	6–17	2	2485
	3.98	1.24	64,000	2	2285	7–29	2	2090
	3.98	2.00	64,000	2	2360	7–10	2	2475
3500	3.98	0	55,200	2	3605	5–29	2	3810
	3.98	1.24	79,000	2	3400	7–15	2	3490
	3.98	2.00	79,000	2	3965	6–23	2	3945
5000	3.98	0	72,000	2	4685	6– 5	2	4515
	3.98	1.24	96,000	2	5310	7–21	2	5320
	3.98	2.00	96,000	2	5435	6–30	2	5390
2000	0	0	0	..	....	Sept.	4	2110
3500	0	0	0	..	....	Sept.	6	3590
5000	0	0	0	..	....	Sept.	4	5610

being made on all of the columns throughout the year using a 10-in. Whittemore strain gage.

Table 1 shows that three grades of concrete were used. These were designed to produce 56-day cylinder strengths of 2000, 3500, and 5000 lb. per sq. in. Three percentages of longitudinal reinforcement, roughly 1.5, 4 and 6, and two percentages of spiral, 1 and 2, were employed in the various columns.

In addition to the 108 reinforced columns originally scheduled 45 plain columns have been made and six of these will be held under constant load for a year. These plain columns were added to provide information on the flow and shrinkage of unreinforced concrete, and to serve as control specimens with regard to the strength of the concrete in the reinforced columns at the end of a year. Four 6 by 12-in. control cylinders were also made with each column, two to be tested at the age of 56 days, and two to be tested a year later when the column is tested to failure.

3. *Materials.*—Details regarding the properties of the concrete and reinforcing steel and a description of the making of columns are given in the progress report published in the A. C. I. JOURNAL, February, 1931.

Average properties of the three concrete mixtures employed are as follows:

Designed strength, lb. per sq. in. (56-day moist curing)	2000	3500	5000
Proportions of mix, by weight	1:4.05:4.95	1:3.27:3.98	1:2.25:2.75
Water-cement ratio, by volume	1.40	1.10	0.80
Average slump, in.	2.9	4.3	5.1

The actual 56-day cylinder strengths are given in Table 1. The materials used in all columns were Lehigh portland cement, Lincoln, Ill. sand and Attica, Ind. gravel. The concrete was mixed 4 minutes in a one-bag batch mixer.

Both the longitudinal and spiral reinforcement were of intermediate grade steel. The selection of bars to provide the three percentages of longitudinal steel was as follows:

1.57 per cent	4 ½-in. round	Yield Point, 45,600 lb. per sq. in.
3.98 per cent	8 ½-in. square	Yield Point, 53,400 lb. per sq. in.
5.94 per cent	4 ⅝-in. round 4 ¾-in. round	Yield Point, 39,300 lb. per sq. in. 51,100 lb. per sq. in.

The reinforcing spirals, of 8-in. outside diameter, were made of hot-rolled rod. The 1.24 per cent spirals were of No. 5 rod, 1.35 in. pitch, and the 2.0 per cent spirals were of ¼-in. rod, 1.19 in. pitch. The stress at which a unit elongation of the spiral rod of 0.5 per cent was produced (in tension tests of coupons) has been taken as a "useful limit" for the material; the average values found were 49,400 lb. per sq. in. for the No. 5 rod and 48,200 lb. per sq. in. for the ¼-in. rod. The respective ultimate tensile strengths were 79,500 and 74,200 lb. per sq. in.

4. *Making and Curing Test Columns.*—Preparatory to making columns, the reinforcing bars were milled to exact length and wired to the spirals or ties to form a unit. The columns were cast in tight steel forms. A tamping rod and a light air hammer were used in placing the concrete. Precautions were taken to bring both ends of the reinforcing bars into the planes of the concrete bearing faces of the columns.

Forms were removed 24 hours after a column was cast. Due to the fact that the forms were water-tight and the top of the column had been covered by a metal capping plate, the column surfaces were damp when the forms were removed and there had evidently been little or no loss of moisture from the column. Shortly after the removal of forms steel plugs were set to form gage lines on which to measure concrete deformations, and gage lines were laid out on the longitudinal steel. The column was then placed in the moist room or stored under wet burlap for one day, when all gage lines were drilled and initial strain measurements were taken. The exposed steel at gage holes was then given a coating of vaseline as a protection against corrosion and the columns were placed in the moist room to complete the 56-day period of moist curing. Strain measurements taken at the end of this period furnished a record of the volume change, usually a slight expansion, produced by about 54 days of moist storage.

5. *Method of Applying Continuous Loads.*—The apparatus for applying and holding a constant load on the columns may be described by reference to Fig. 1. Companion columns, tested in pairs to save apparatus, are confined between bearing plates by four steel tie rods. The desired tension is maintained in these

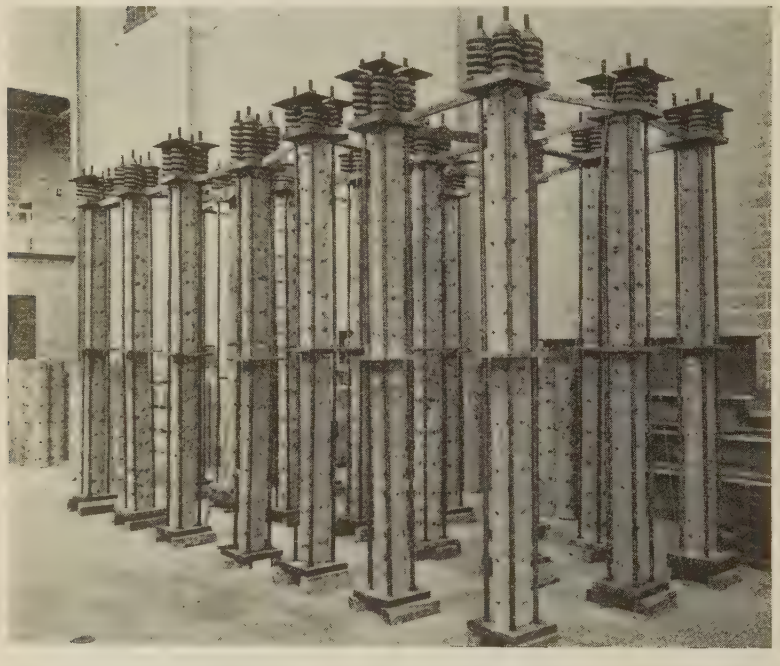


FIG. 1—VIEW OF COLUMNS OF SERIES 3 IN MATERIALS TESTING LABORATORY

rods by four heavy coiled compression springs, to be seen at the top of each rig in the figure. These springs were calibrated, after having been subjected to ten repetitions of their capacity load (for the purpose of relieving localized stresses and thus minimizing the permanent set which might occur under the long continued loading). The desired load was applied to the columns by tightening the nuts on the four tie-rods until the necessary shortening of each spring was reached. This shortening varied from 1.0 to 1.3 in. with the five sizes of springs used, and since errors in measurement on the two gage lengths on each of the four springs would normally be compensating, the design loads are believed to be accurate to within 1 per cent. A complete set of strain measurements was taken on the columns immediately before and after applying the load, and at regular intervals thereafter.

Measurements on gage lines on the springs were made coincident with the reading of strains in the columns. With shrinkage and yielding of the columns a slight decrease in the indicated spring load resulted. One complete adjustment of all loading rigs was made after about three months of loading and since that time there has been little change in the applied load. At the time of the adjustment, several springs were removed and checked, but practically no permanent set of the springs could be detected.

It was anticipated that unequal strains might be produced on opposite sides of a column by eccentricity of load or by variations in the material of the column. To minimize eccentric loading and to stabilize the loading rig, the tie rods were run through holes in a bearing plate between columns at mid-height of the rig. There has been no appreciable evidence of bending or unequal yielding.

The tie-rods and plates shown in Fig. 1 were furnished for the work through the courtesy of the American System of Reinforcing and the Illinois Steel Co., Chicago.

6. *Working Loads.*—As previously noted, the columns were loaded at safe working stresses computed by use of the design formulas for tied and spirally reinforced columns of the A. C. I. Joint Building Code and for spirally reinforced columns of the 1916 New York City building code. This does not imply that the test columns met all of the design requirements of these two codes. For the purposes of the investigation it was considered desirable to use combinations of materials outside the restricted field of either code. Thus most of the columns have a percentage of spiral reinforcement in excess of one-fourth that of the longitudinal reinforcement, as required by the A. C. I. code, and one type of column has a proportion less than one-fourth. None of the columns have an appreciable protective shell of concrete outside the reinforcement. The New York code does not recognize a grade of concrete having a strength of 3500 lb. per sq. in., nor does it permit the use of more than 4 per cent of longitudinal reinforcement. Hence the use of these formulas in determining working loads is not in strict accordance with the apparent intent of the codes, but it is thought that the use that has been made of them has been reasonable and fairly conservative.

The working loads used with all columns are listed in Table 1. The loads for columns without spiral reinforcement were computed by use of the Joint Building Code formula for tied columns. For the plain concrete columns, no formula is applicable. The loads used were chosen to produce concrete stresses comparable with those existing in the reinforced columns in the early stages of loading. It is evident that further tests are needed to determine the flow of the different grades of concrete under various intensities of load.

7. *Flow and Shrinkage During a Period of Five Months.*— The end of the one-year loading period will be reached for the various reinforced columns during June, July and August, 1931. Similar periods for plain columns will extend to March, 1932. However, observations have been taken on all of the reinforced columns and some of the plain ones for five months or more, and it appears that the most rapid development of flow and shrinkage strains has taken place in this period. Figs. 2 to 9 show values of measured strains in the columns, beginning at the end of the 56-day curing period, when the columns were placed under load, and continuing for 5 months. Using a modulus of elasticity of 30,000,000 lb. per sq. in. these strains may be translated into stresses in the longitudinal steel. The curves have been grouped as much as possible according to the class of concrete used in the columns. Generally the time-deformation curves for each pair of companion columns is plotted on the same diagram with similar curves for the corresponding unloaded columns. The storage of columns in the air of the laboratory is designated as "Air Storage" while storage in the moist room is designated as "Moist Storage."

It may be noted that the observations plotted on these diagrams (excepting Fig. 9) represent the strains in reinforcing bars only. The strains on adjacent concrete gage lines have not yet been computed from the original readings, but it may be assumed that they will closely approximate those in the longitudinal steel.

A record was kept of temperatures and relative humidity in both the laboratory and moist rooms when all strain observations were taken. A recording thermometer in the moist room has

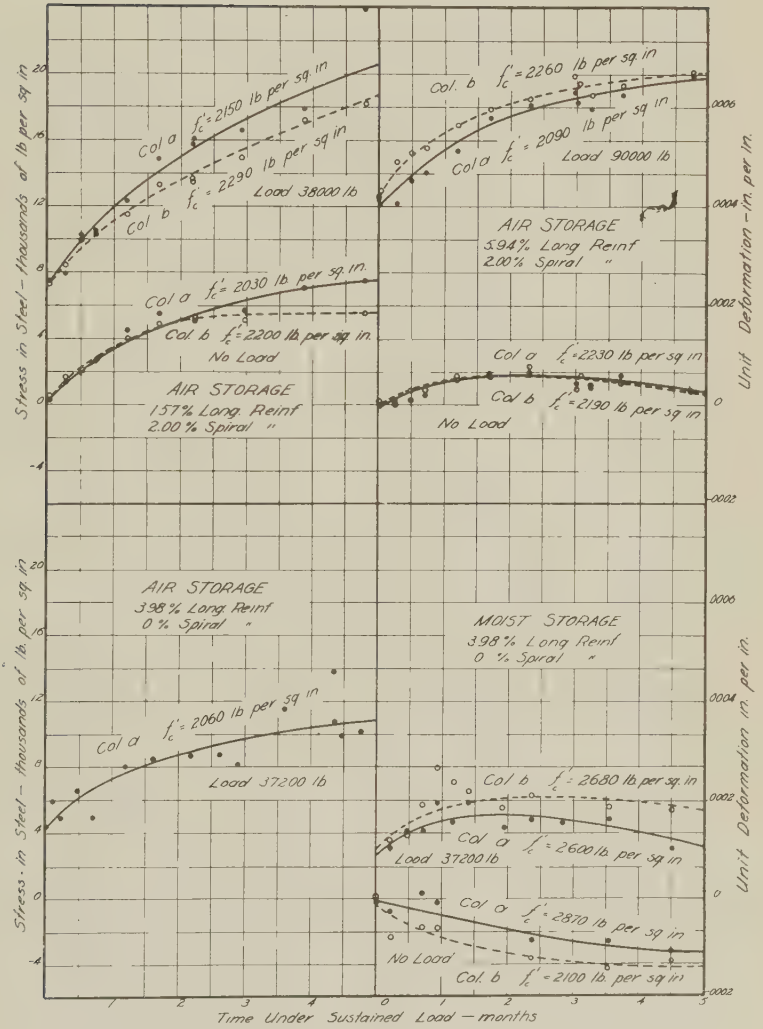


FIG. 2—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 2000-LB. CONCRETE

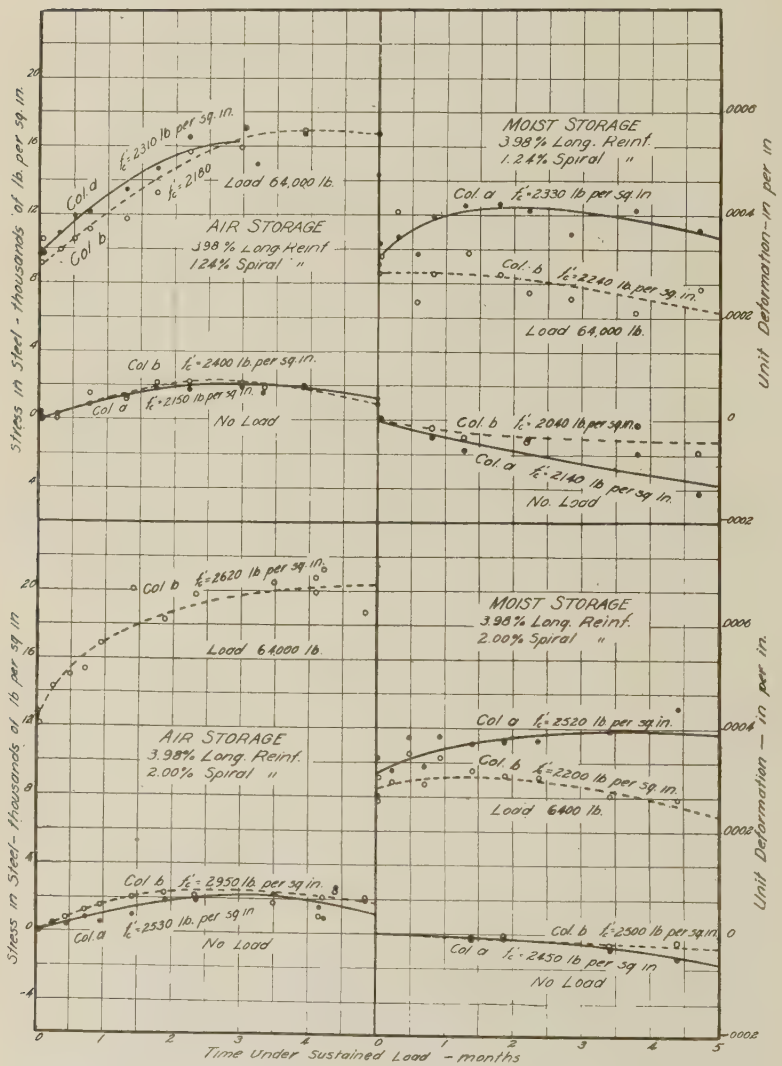


FIG. 3—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 2000-LB. CONCRETE

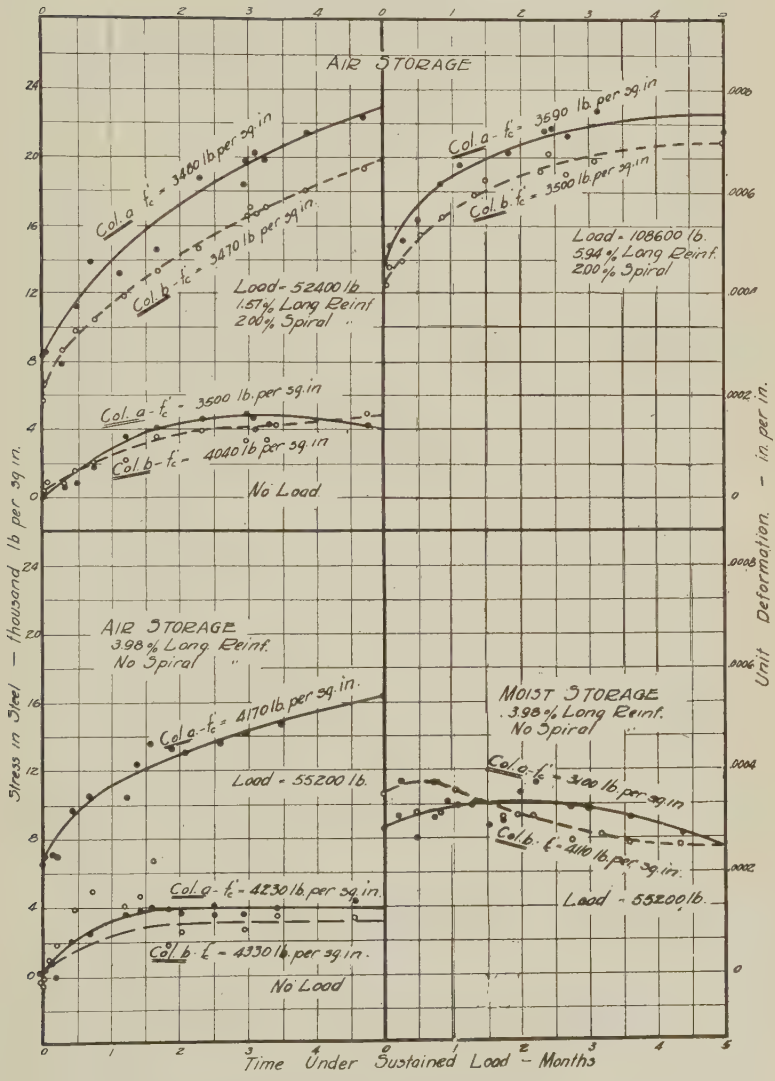


FIG. 4—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 3500-LB. CONCRETE

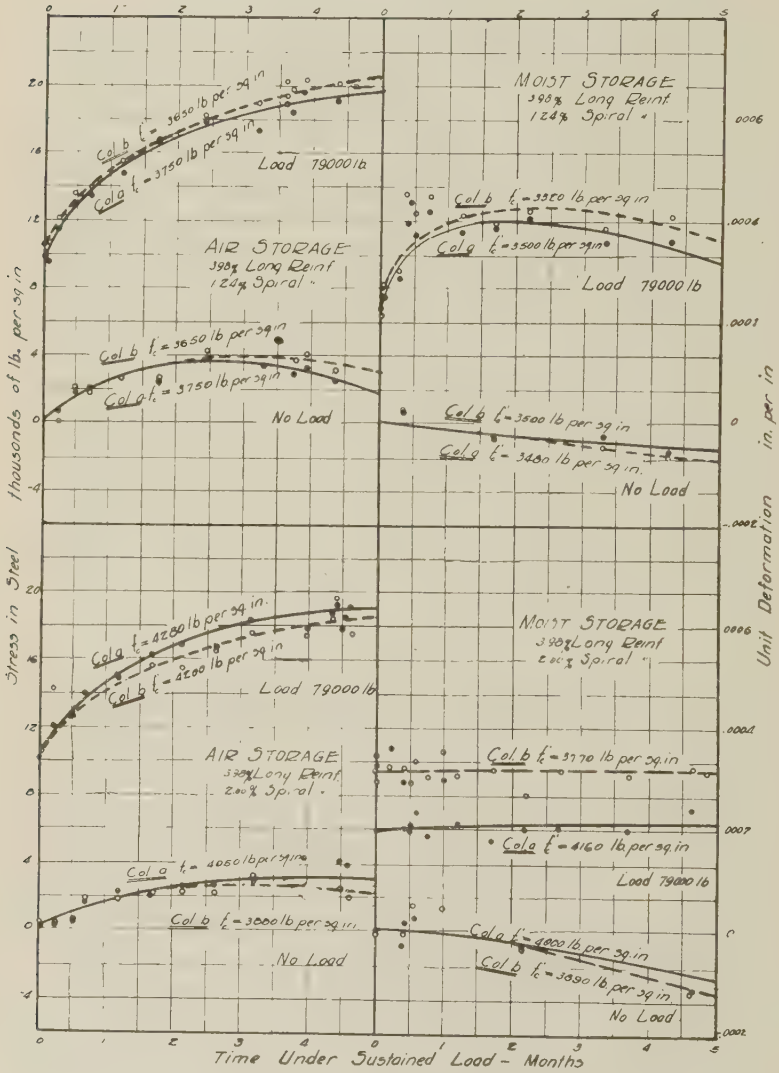


FIG. 5—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 3500-LB. CONCRETE

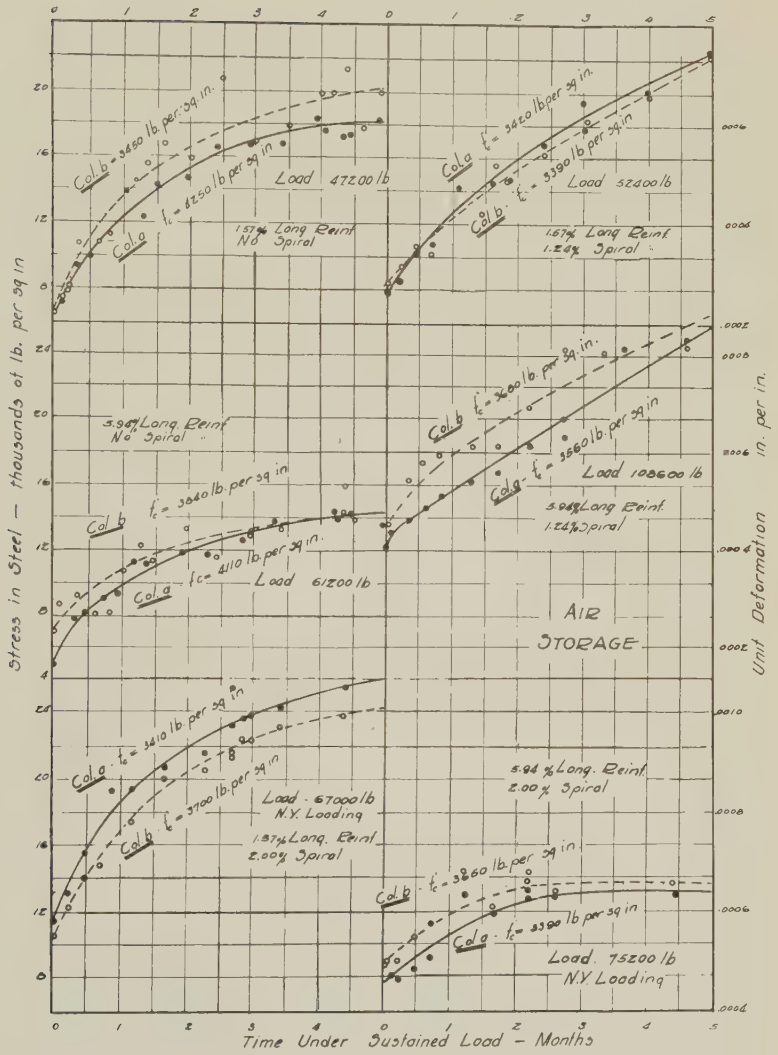


FIG. 6—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 3500-LB. CONCRETE

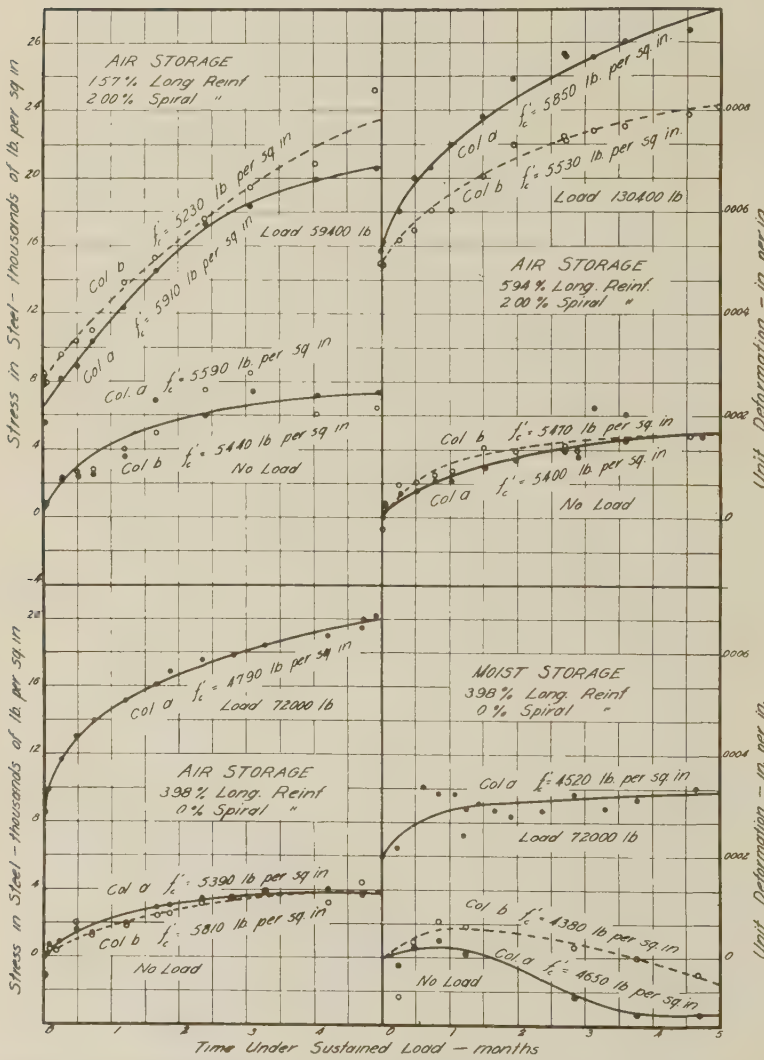


FIG. 7—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 5000-LB. CONCRETE

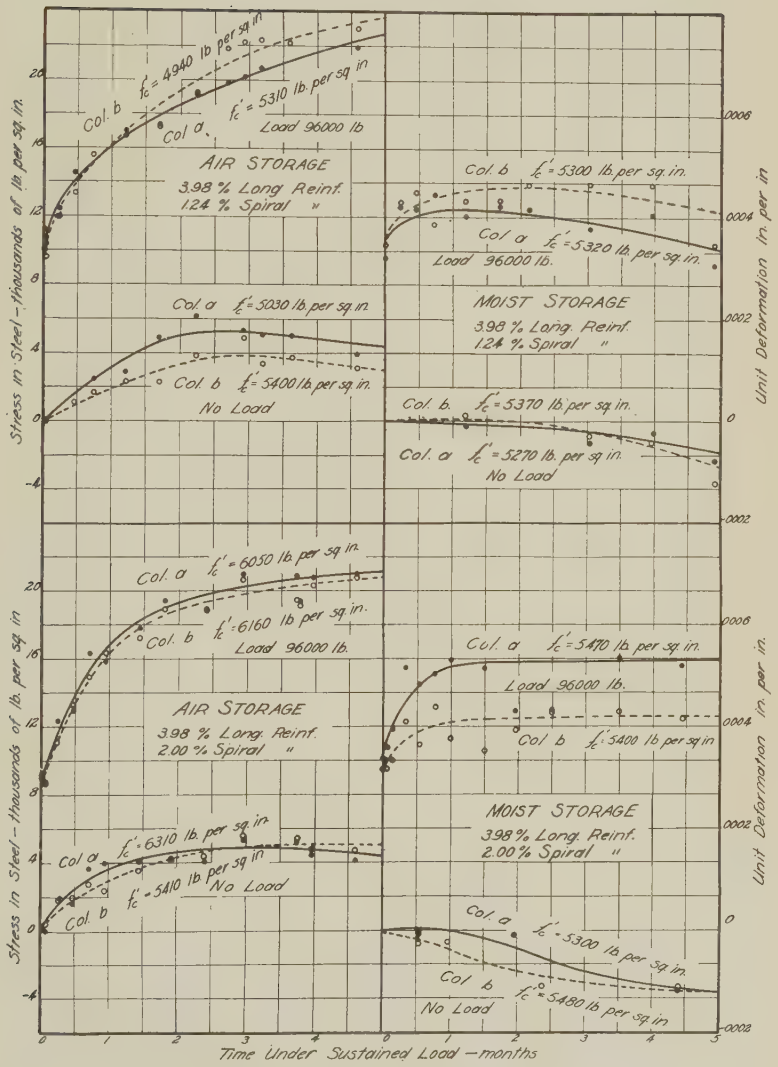


FIG. 8—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 5000-LB. CONCRETE

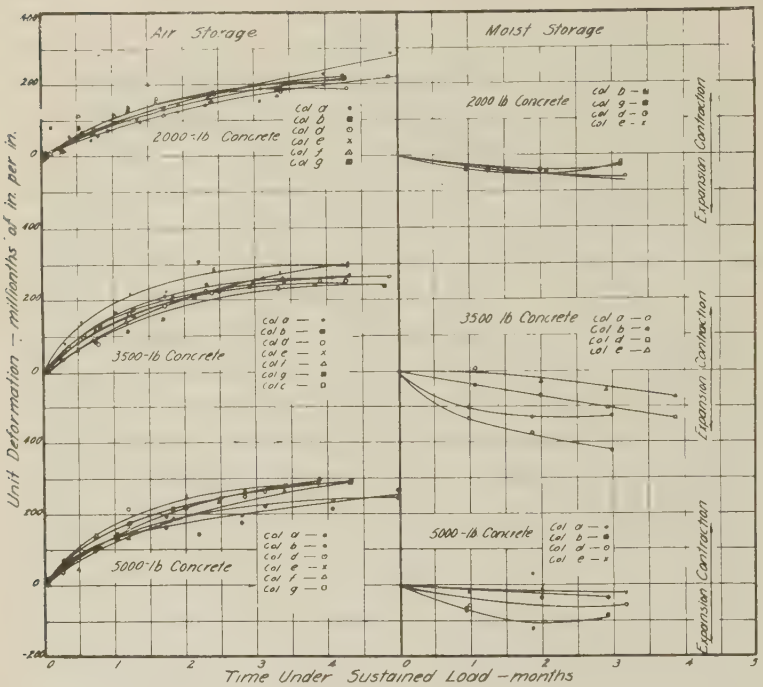


FIG. 9—TIME-DEFORMATION CURVES FOR PLAIN CONCRETE COLUMNS

provided a continuous temperature record over the period of the tests, and a recording psychrometer was installed in the laboratory storage room in January, 1931. Previous to this date, the temperature and humidity records for the laboratory storage room were mainly individual day time observations. In Fig. 10 is shown the record of the average recorded daily temperatures for both dry and moist storage rooms since June, 1930. A similar record of the average relative humidity of the laboratory air is given in the figure. From many observations it is known that the relative humidity of the moist room was maintained at practically 100 per cent, except for short intervals when strain measurements were taken.

The range of temperatures and humidities may be of interest in connection with the curves of Figs. 2 to 9. Knowing from

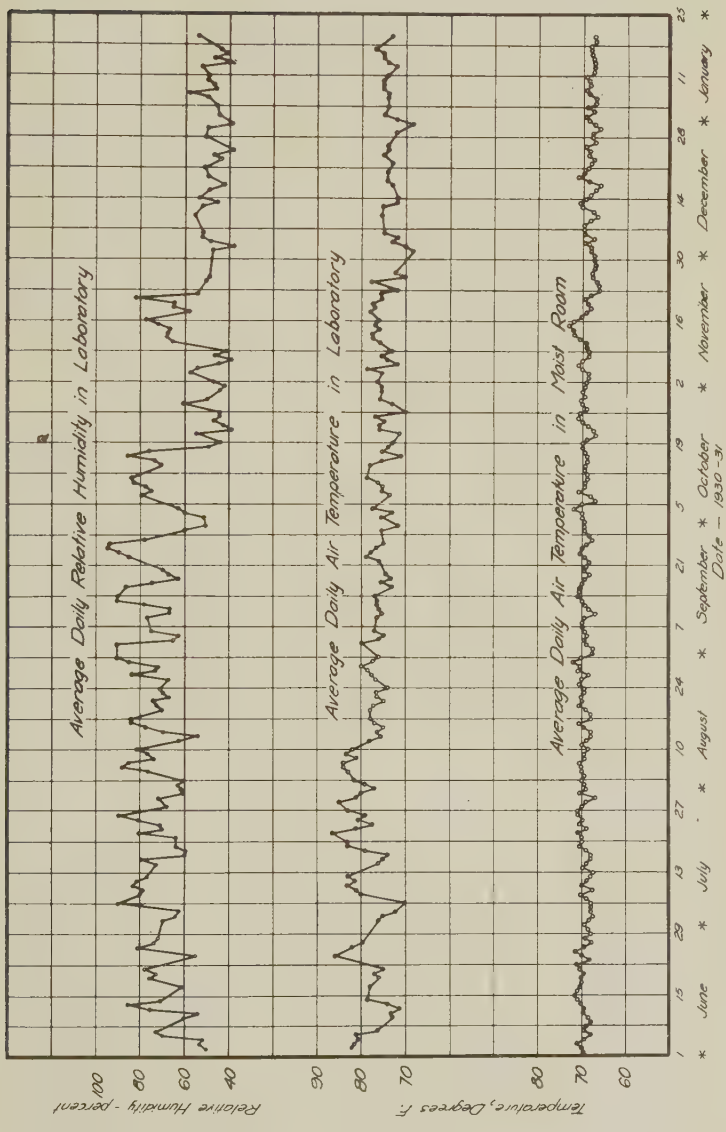


FIG. 10—TEMPERATURE AND HUMIDITY RECORDS FOR PERIOD OF TESTS

Table 1 the date at which loading of any column was started, the trend of temperature and humidity variations thereafter may be traced in Fig. 10, and compared with any unusual variation in the measured strains in the column.

The strains plotted in Figs. 2 to 9 have not been corrected for the effect of temperature changes. For the columns in moist storage such temperature changes were negligible. The air-stored columns are known to have been at very nearly 70 deg. F. when removed from moist storage and placed under spring loading, and subsequent air temperatures of the laboratory are given in Fig. 10; however, it is certain that the effective temperature of any column did not vary as widely or as rapidly as the air temperature of the laboratory. A correction in column strains based on the variation in air temperature might introduce errors as great as if no correction were attempted. It is believed that the uncorrected strains in Figs. 2 to 9 are in no case in error due to temperature effects more than 0.000067 (corresponding to an increase of 10 deg. F. over the initial temperature of the column, and to an indicated tensile stress in the steel of 2000 lb. per sq. in.) and for observations made since August, 1930, the possible error would be much less than this. Any such error is opposite in sense to the effect of shrinkage or flow; if a column has increased in temperature during the loading period, the measured strains attributed to shrinkage and flow are less than the true amount of shrinkage and flow.

The amount of plastic flow, obtained by subtracting the strains of an unloaded column from those of the corresponding loaded one, is unaffected by any temperature changes, since such changes are the same in loaded and unloaded columns.

The strain readings were taken at the time of loading the columns, then generally after 1, 3, 7, 14 and 28 days, and at monthly intervals thereafter. The strain measurements were taken on a total of 16 gage lines, 8 at the middle and 4 each near the top and bottom of column, and included two independent sets of observations. These measurements on a total of 153 columns have required the help of several observers, and in general the strain measurements for any column were taken by two or more observers. The accuracy of the strain gage measurements in this series of tests is therefore considerably less than that obtained

in the other series of the investigation, in which a complete test could be made by one observer in a period of only a few hours, with a definitely established routine for strain measurements.

8. *Elastic Deformations.*—The deformation measured at the time the spring load was applied to any column will be denoted here as an elastic deformation, in contra-distinction to the deformation which resulted from continued loading. Since the concrete stresses were generally well below one-half of the compressive strength, and the steel stress only a small fraction of the proportional limit, it is likely that the strains developed in applying the load were very nearly, if not wholly, elastic. The corresponding steel stresses, which fix the starting point of the curves of Figs. 2 to 8, seem rather erratic from an inspection of the figures. However, a little study of the probable elastic action of the various columns tends to verify the general trend of the measured values. It seems reasonable to assume that the initial stresses due to a load no greater than the design load will be determined by the elastic properties of the two materials; that is, the stresses in concrete and longitudinal steel will be proportional to the respective moduli of elasticity. A large number of extensometer tests of the control cylinders of this series have been made, but not analyzed; however, the average moduli for 10 cylinders, selected at random, of each grade of concrete are as follows:

Designed Strength lb. per sq. in.	Actual Strength lb. per sq. in.	Modulus of Elasticity lb. per sq. in.	Modular Ratio n
2000	2200	2,830,000	10.6
3500	3730	3,800,000	7.9
5000	5460	4,290,000	7.0

These moduli represent the initial slopes of the stress-strain curves, for which the proportional limit was usually 30 to 50 per cent of the ultimate strength. Using 30,000,000 lb. per sq. in. as the modulus for steel, values of the ratio, n, of the moduli for steel and concrete have been computed. This ratio should also represent the ratio of initial steel and concrete stresses. Table 2 gives values of the calculated initial stresses in steel and concrete due to the design loads, based upon the above values of n. The table also shows values of the stresses determined from

TABLE 2. STRESSES IN COLUMNS OF SERIES 3.

The unit stresses, f_s in steel, and f_c in concrete, are in lb. per sq. in. Positive values are compressive stress; negative values, tensile stress.

Nominal Column Design			Design Load lb.	Loaded Columns							Unloaded Columns	
	Percentage Reinforcement			At Load Application					After 5 Mo.		After 5 Mo.	
Concrete Strength Lb. per sq. in.	Vertical	Spiral		From Elastic Theory		From Strains			From Strains		From Strains	
				f_s	f_c	f_s	f_c	f_s/f_c	f_s	f_c	f_c	f_s
Air Storage												
2000	1	2	38000	6900	660	7300	650	11.2	19600	455	5600	— 89
	4	0	37200	5700	535	4300	685	6.3	10800	325	....	....
	4	1	64000	9800	920	9400	940	10.0	16000	665	1200	— 50
	4	2	64000	9800	920	12100	825	14.7	20400	480	1600	— 66
	6	2	90000	12100	1140	12200	1130	10.8	20400*	610	800	— 50
3500	1	0	47200	6750	845	6400	855	7.5	19100	650	....	....
	1	1	52400	7500	940	9800	905	10.8	20000	740	....	....
	1	2	52400	7500	940	7000	945	7.4	21100	725	4300	— 68
	1	2	**67000	9550	1205	11100	1175	9.5	25500	945	....	....
	4	0	55200	6850	6500	6500	870	7.5	16300	470	3400	—140
	4	1	79000	9800	1235	9800	1230	7.9	20000	810	2600	—108
	4	2	79000	9800	1235	10200	1210	8.5	19000	850	2800	—116
	6	0	81200	9100	1145	6000	1335	4.5	13700	850	....	....
	6	1	108600	12050	1540	12400	1515	8.2	24800	730	....	....
	6	2	108600	12050	1540	13100	1470	8.9	21600	930	....	....
	6	2	**75200	8420	1060	8300	1065	7.8	13400	740	....	....
5000	1	2	59400	7650	1080	7300	1080	6.7	22100	850	7200	—114
	4	0	72000	8150	1155	8600	1130	7.6	20000	660	3900	—161
	4	1	96000	10850	1540	10600	1550	10.1	23000	1040	3700	—153
	4	2	96000	10850	1540	9000	1620	5.6	21100	1115	4700	—194
	6	2	130400	13300	1915	15500	1780	8.7	27100	1045	5000	—316
Moist Storage												
2000	4	0	37200	5700	535	4200	595	7.1	10900	325	—3600	149
	4	1	64000	9800	920	9200	945	9.8	8500	975	—2500	104
	4	2	64000	9800	920	9000	950	9.5	9000	950	—1100	45
3500	4	0	55200	6850	860	9600	745	12.9	7500	830	....	....
	4	1	79000	9800	1235	7600	1320	5.8	10100	1210	—1800	75
	4	2	79000	9800	1235	7600	1320	5.8	7900	1310	—3000	124
5000	4	0	72000	8150	1155	6000	1240	4.8	10000	1080	—2400	100
	4	1	96000	10850	1540	11300	1520	7.4	11000	1530	—2200	92
	4	2	96000	10850	1540	9800	1580	6.2	14000	1410	—3700	153

**According to N. Y. City Bldg. Code; all others according to A. C. I. Joint Bldg. Code.
*By extrapolation.

strain measurements. It is of interest that although there are a few rather large discrepancies between the two sets of values, the general trend of the two is quite similar. Furthermore, the average values of the ratio of the measured initial steel and concrete stresses, for columns of the three grades of concrete are 10.0, 8.1, and 7.1, as compared with the above values for n (based on cylinder tests) of 10.6 ,7.9 and 7.0.

The design loads which were computed by the A. C. I. formula for spirally reinforced columns are seen to produce relatively high initial concrete and steel stresses in the columns having a large percentage of longitudinal steel. The ratio of initial concrete stress to the concrete strength is not constant, but in general increases with an increase in percentage of longitudinal reinforcement and decreases with an increase in concrete strength.

Table 2 includes data on the stresses in concrete and steel after a 5 months period of observation for both loaded and unloaded columns. The stress in the unloaded columns may be attributed to the effect of volume changes, while the increase in stress in the loaded columns is evidently due to volume change, plastic flow and change in elastic strain. It is interesting to note that the greatest increase in steel stress due to the three combined effects was 14,800 lb. per sq. in., and in most cases with the air-stored columns the increase was from 6000 to 14,000 lb. per sq. in. For the moist-stored columns, the increase was very much less.

9. *Relation Between Strength of Concrete and Column Deformations.*—For further study of the effect of variations in concrete strengths upon column deformations, curves from Figs. 2 to 8 have been replotted in Fig. 11. Each curve represents the average result from a pair of companion columns and each group of curves represents the results from columns that are alike except for the quality of concrete with which they were made. It is difficult to make comparisons as to the relation between concrete strength and the deformation due to flow and shrinkage because the concrete in the three types of column was subjected initially to neither the same stress nor to the same proportion of the concrete strength in the three cases. Furthermore, for air-stored columns, the stress in the concrete gradually decreased as the concrete deformed so that at the end of the 5-month period the concrete stress had lowered considerably, as shown in Table 2. However, the combined effect upon column action is such that there is no great difference apparent in the time-rate of deformation for the columns made with 2000, 3500 and 5000-lb. concrete. It seems probable that there may be a relation between intensity of concrete stress and the corresponding time-rate of deformation

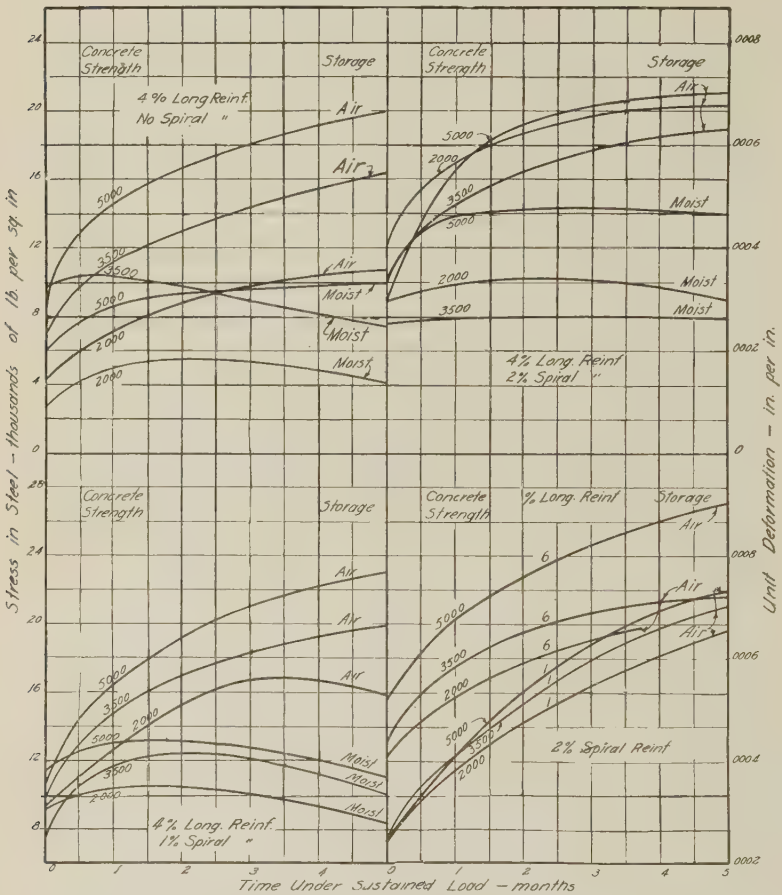


FIG. 11—TIME-DEFORMATION CURVES, VARYING CONCRETE STRENGTHS

at any point on the curves of Fig. 11. Such a relation would be expected to differ for the three grades of concrete.

Fig. 11 also permits a comparison between the deformations found in columns held under load in air and in moist storage. The deformations in the air-stored columns were much larger than those in the moist-cured columns and were increasing at the end of 5 months, whereas in many cases the strains in the moist-stored columns had reached a maximum and were decreas-

ing at the end of the period shown. Such a decrease is also shown by the strain curves for unloaded columns in Figs. 2 to 8 which would indicate that the columns were expanding due to the 5 months storage in the moist room. Since the total deformation of loaded columns is evidently a combination of shrinkage or expansion, plastic flow and elastic deformation produced by the applied load, it seems plausible that the plastic flow after several months of moist storage may be counteracted by the expansion due to the presence of moisture and by an increase in the stiffness of the concrete due to the moist curing. That the elastic strain may decrease due to an increase in modulus of elasticity with age has been shown by Glanville[4] in recent tests of flow of plain concrete under sustained load. This may explain the fact that the flow of plain concrete has been found by Davis[5] and others to be less for specimens in moist storage than for those in dry storage.

10. *Relation Between Percentage of Spiral Reinforcement and Column Deformations*—To permit convenient comparison of time-deformation curves for columns that are alike except for the percentage of spiral reinforcement, Fig. 12 has been drawn. Each pair of curves represents average results for columns with 1.24 and 2.0 per cent of spiral reinforcement, for which the design loads were identical. It was planned to include the results from the tied columns in the comparison, but since the design loads differed from those for spirally reinforced columns, the results did not appear comparable. There is no reason to believe that the presence of spiral reinforcement should have any appreciable effect on the longitudinal strains due to shrinkage, flow or load of the intensity used; this view is supported by the evidence given in Fig. 12. There is no consistent difference in the deformations developed in the presence of the two amounts of spiral, and the differences are of a magnitude which may be explained by errors of observation and differences in the materials and treatment of the columns.

11. *Relation Between Percentage of Longitudinal Reinforcement and Column Deformations.*—Average time-deformation

[4] "The Creep or Flow of Concrete under Load," by W. H. Glanville, Building Research Technical Paper No. 12. Department of Scientific and Industrial Research, London, 1930.

[5] "Flow of Concrete Under Sustained Compressive Stress." by R. E. Davis and H. E. Davis, Proc. A. S. T. M., V. 30, Part 2, 1930.

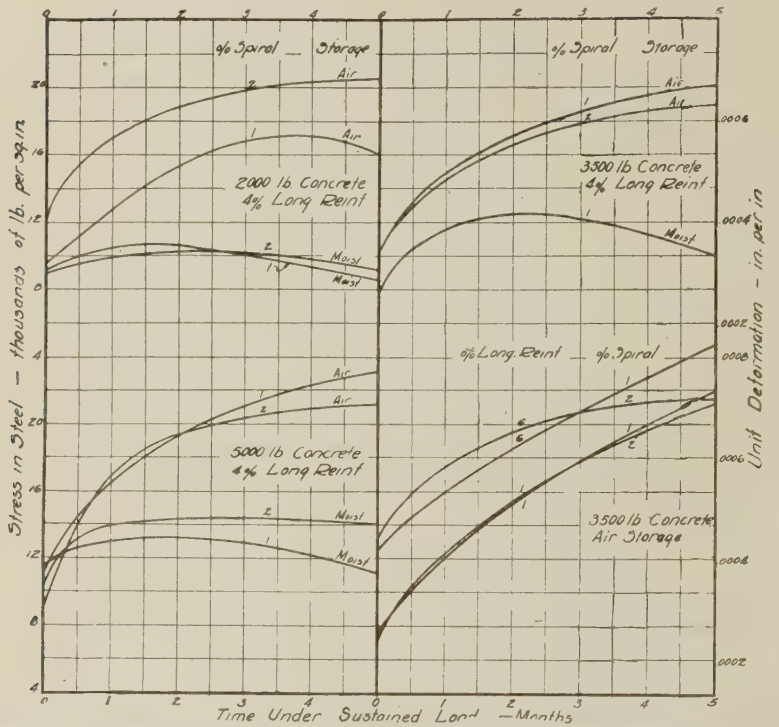


FIG. 12—TIME-DEFORMATION CURVES, VARYING PERCENTAGES OF SPIRAL REINFORCEMENT

curves from Figs. 2 to 8 have been grouped in Fig. 13 to facilitate a study of the effect of variations in amount of longitudinal reinforcement in columns otherwise alike. The three percentages of longitudinal steel were roughly 1.5, 4 and 6. Since all moist-stored columns were made with 4 per cent of longitudinal steel, this comparison is made only between the results from columns stored in air. The figure includes curves for columns made with three grades of concrete and 2.0 per cent spiral reinforcement, also for columns of 3500-lb. concrete with 0, 1 and 2 per cent of spiral reinforcement.

As noted with reference to Table 2, the curves for columns with 6 per cent of longitudinal steel generally start at a higher steel stress than do those for the smaller percentages. This follows from the fact that the A. C. I. formula for spirally rein-

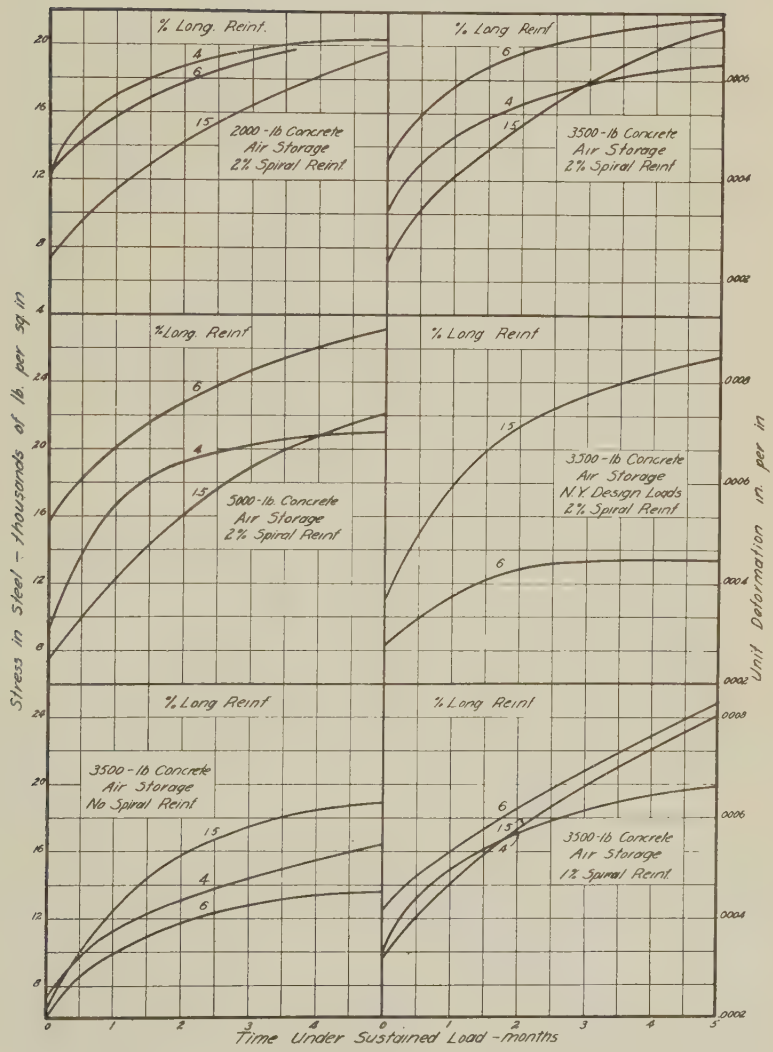


FIG. 13—TIME-DEFORMATION CURVES, VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT

forced columns, used in determining the applied loads, permits an increased working stress in both concrete and steel as the amount of longitudinal reinforcement is increased. The elastic strains, therefore were to some extent fixed by the formula used, and by the modulus of elasticity of the concrete.

The change in deformation, and in the steel stresses, in 5 months was generally least for the columns with 6 per cent reinforcement, slightly more for those with 4 per cent and quite obviously greatest for those with 1.5 per cent. This may be due

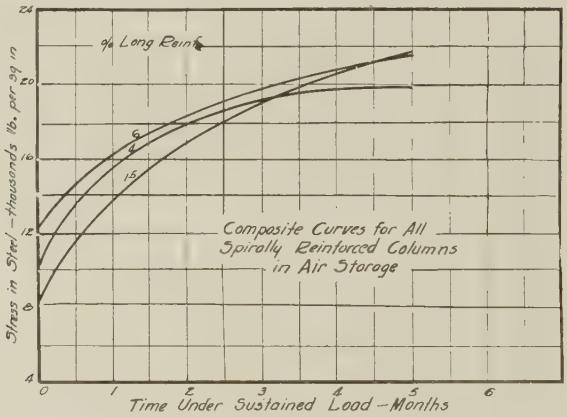


FIG. 14—COMPOSITE CURVES FOR SPIRALLY REINFORCED COLUMNS HAVING VARIOUS PERCENTAGES OF LONGITUDINAL REINFORCEMENT

to the fact that a given amount of yielding or flow decreases the concrete stress much more rapidly in columns with a large amount of longitudinal reinforcement than in those with a small amount. There is no marked difference in the appearance of the curves due to variation in concrete strength or in percentage of spiral reinforcement.

To summarize the relation between longitudinal reinforcement and column deformations, Fig. 14 has been drawn by averaging together the results of all air-stored spirally reinforced columns containing each percentage of longitudinal steel, disregarding the concrete strength and the amount of spiral reinforcement. Each of the composite curves of Fig. 14 thus represents the average

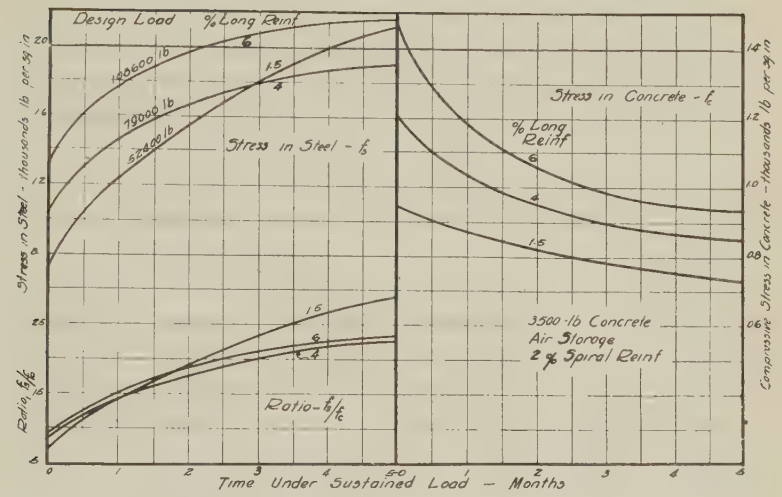


FIG. 15—CURVES SHOWING REDISTRIBUTION OF STRESSES IN CONCRETE AND STEEL DURING A 5-MONTH LOADING PERIOD.

results from ten or more columns. The rate of deformation, and of increase in steel stress during the first two weeks of loading is about equal for the lower two percentages of reinforcement but later on the curve for 4 per cent of reinforcement shows a rapid decrease in rate of deformation as compared to the other two. The curve for 1.5 per cent of reinforcement which started well below the other two, had crossed both of them after 4½ months of loading, and at the end of the 5-month period, shows a much greater rate of increase than the other two curves.

It may be noted that the increase in steel stress during 5 months under load as shown by the composite curves of Fig. 14 were as follows: With 6 per cent of longitudinal reinforcement, 9200 lb. per sq. in.; with 4 per cent, 9700 lb. per sq. in.; and with 1.5 per cent, 13200 lb. per sq. in.

12. *Redistribution of Stresses in Concrete and Steel Due to Inelastic Deformation of Concrete.*—The various time-deformation curves have indicated the increase in the stresses in the longitudinal reinforcement due to shrinkage and flow of the concrete, and it is evident that as the total stress in the steel increased, the total stress in the concrete decreased an equal amount. Table

2 gives a summary of the measured steel stress produced in all columns due to the design load and the increased value of this stress after a 5-months period of loading. From the difference between the applied load and the total stress in the longitudinal steel, the unit stresses in the concrete have also been found, as given in the table.

A conception of the variation in both steel and concrete stresses with time may be aided by reference to Fig. 15. The curves for stress in steel are similar to those of preceding figures, and the corresponding curves showing the concrete stresses at any time during the 5 months have been determined by computation as described above. It is evident that as yielding of the concrete progressed, the concrete stresses fell off rapidly; on the other hand at considerably decreased concrete stresses, yielding was still progressing. Various tests of plastic flow of plain concrete are in agreement on the point that under a constant load the rate of flow gradually decreases with time; if, as in these columns, the applied stress also decreases it would be expected that the plastic flow would progress at a very rapidly diminishing rate. However, tests have been made in which the flow apparently continued for several years, so that it is unsafe to draw any conclusions now as to possible limits in the amount of flow still to be expected.

The lower curves of Fig. 15 show the way in which values of the ratio of steel and concrete stresses varied over the 5-month period. The ratio may be considered equivalent to the modular ratio, n, wherein the modulus of elasticity used is the "sustained modulus of elasticity" and includes the effect of shrinkage, flow and elastic deformation. The curves show no great difference in values of the ratio for the three amounts of reinforcement used, but do show that the values after 5 months of loading average 3 times as great as the initial values. The initial values are in good agreement with the value of n used in computing the elastic distribution of stress in Table 2.

13. *Effect of Initial Strains on Load-Carrying Capacity of Column.*—It is hoped that the completion of this series of tests will furnish information on a number of questions, as yet unanswered, such as: (1) Will the stresses induced in the reinforcing steel reach the yield point of the material, and if so, what will be

the effect upon stresses and deformations, (2) Will the presence of large initial steel strains affect the ultimate strength, or in other words, the factor of safety of the column, and (3) Will the column deformations under continued working loads produce undesirable distortions in a structure?

In the present tests the steel stresses after 5 months are in only one case greater than one-half of the yield point stress for the longitudinal steel, and the rate of yielding is decreasing rapidly. It does not appear probable that the yield point will be reached in any column in the one-year loading period. This is on the assumption that the creep of the reinforcing steel under continued load is negligible at stresses below the yield point as ordinarily defined.

It is known that when a steel bar is stressed beyond the yield point in compression, little lateral force is required to prevent a side-wise buckling if the bar is kept straight. The tests of spirally reinforced columns in the other series of the investigation showed that the longitudinal bars carried load effectively beyond the yield point, though large longitudinal deformations of both concrete and steel took place. The plastic deformation of intermediate grade steel at the yield point is very rapid as compared to the time yields for concrete shown in these tests. Hence it appears that if the yield point of the steel is reached, either due to flow of concrete or to an increase in load, the yielding of the steel will tend to increase the stress in the concrete. Whether, under a constant load, the further yielding of both steel and concrete would be sufficient to bring the spiral reinforcement into action is problematic. However, in the case of the columns which are to be loaded to failure after the end of a year, it seems likely that when the steel reaches its yield point, additional load will be taken by the concrete until it reaches a stress corresponding to the strength of the plain concrete, whereupon the spiral reinforcement will be brought into action. If this is correct, the strength of the column should not be appreciably affected by the initial stresses due to flow and shrinkage.

The deformations in the test columns have not exceeded 0.0009 in. per in., corresponding to a steel stress of 27,000 lb. per sq. in. It does not appear that deformations of this magnitude

should cause any structural difficulties, but if the yield point of the steel were exceeded, the deformations might be of considerable importance.

14. *Summary.*—Little can be stated in the way of conclusions regarding this series of tests, since much of the significance of the results will depend upon the behavior of the columns when tested to failure. It may also be added that in the preparation of this report there has been little time to check up discrepancies in recorded data or to study the results. The following summary may be made as to the foregoing discussion:

1. The stresses in longitudinal reinforcement of columns when initially placed under a working load agree fairly well with the values calculated by assuming the stresses in steel and concrete to be proportional to the respective moduli of elasticity of the two materials.

2. The increase in steel stress in columns held under working loads for 5 months was much greater for columns stored in the air of the laboratory than for those stored in the moist room.

3. The increase in steel stress for the loaded columns was slightly greater for columns with 4 per cent of longitudinal reinforcement than for those with 6 per cent, and much greater for those with 1.5 per cent than for either of the other two.

4. The greatest measured stress in any of the test columns, due to the combined effect of load, shrinkage and flow was 27,100 lb. per sq. in., an increase of 11,600 lb. per sq. in., over the initial stress due to the load alone.

5. The rate of flow does not appear to vary greatly with different grades of concrete or in the presence of different amounts of spiral reinforcement. Any study of the rate of flow is complicated by the fact that the concrete stress decreases as the flow and shrinkage increase.

6. The results thus far obtained in this series of tests indicate that the final results will have a very definite bearing on the question of the proper method of design of reinforced concrete columns.

Readers are referred to the JOURNAL *for June, 1931, for discussion of the Reinforced Concrete Column Investigation of Committee 105. Such discussion should reach the Secretary by May 1, 1931.*

Second Progress Report on Column Tests Made at Lehigh University

BY W. A. SLATER* AND INGE LYSE†

SERIES 3—COLUMNS UNDER LONG TIME LOADS

1. *Introduction.*—The first Progress Report of Committee 105 on Column Investigations was published in the April JOURNAL, 1930.[1] That report gives the program of the investigation and results of the preliminary tests of the materials. The first progress report from Lehigh University on the column tests gives results for Series 1 and 2 and was published in the February, 1931, JOURNAL.[2] The present (second) progress report from Lehigh University gives results for Series 3 on columns subjected to working loads for 20 weeks. Wherever references are made to the results for Series 1 and 2 the information will be found in the first progress report.

2. *Purpose and Program.*—The purpose of Series 3 is to study the effect of sustained working load on the deformation and strength of reinforced concrete columns. The applied working loads were computed according to the formulas given in the present American Concrete Institute Building Code (1928)[3] and the present New York Building Code (1916).[4] The cases to which

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[1]See Progress Report of Committee 105, *Proc. A. C. I.* V. 26, p. 601, 1930.

[2]See Progress Report on Column Tests at Lehigh University, *Proc. A. C, I.* V. 27, p. 677. 1931.

[3] $\frac{P}{A} = \left(300 + (0.10 + 4p) f'_c\right) \left(1 + (n - 1)\, p\right)$

[4] $\frac{P}{A} = f_c \left(1 + (n - 1)\, p + \frac{2 f_s n'}{f_c}\right)$

in which P = total load on column

A = total sectional area within outer circumference of spiral
f'_c = ultimate strength of control cylinders
f_c = allowable working stress in concrete. (Foot note continued next page)

each formula was applied are indicated in Tables 1 and 6. The columns in this series did not conform, however, with either code so far as the thickness of the concrete covering of the reinforcement was concerned. The minimum thickness of this covering should be 1½ in. according to the Institute Code and 2 in. according to the New York Code. All columns in this series had an average thickness of the concrete covering of only ⅛ in. A further deviation from the New York Code was made for the columns having 6 per cent longitudinal reinforcement. Even though the New York Code specifies a maximum of 4 per cent longitudinal reinforcement, the load applied to two of the columns having 6 per cent reinforcement was computed from the formula given by the New York Code. The formulas are designed to be used in connection with the 28-day compressive strength of the control cylinders, whereas in computing the working loads for these tests the strength at the time of application of load (56 days) was used instead.

These rather wide divergences from the limits imposed by the Codes are justified by the fact that this investigation is aimed at determining trends, and the applicability of the principles underlying the formulas, rather than the mere testing of columns conforming in every respect to the limitations, within which the Codes are necessarily surrounded. For example, the shell provided outside the spiral is generally intended as fire protection, but to include such a shell on these test columns would, to a certain degree, defeat one purpose of the investigation which is to determine the strength of the spiral core.

The program of tests for Series 3 is shown in Table 1. This does not differ from the program for Series 3 given in the 1930 *Proceedings*, page 611, but is given in more detail for the sake of clarity. Three types of concrete, designed for cylinder strengths of 2000, 3500, and 5000 lb. per sq. in. at 56 days, respectively, were included. The reinforcement was all of intermediate grade

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f_s = allowable working stress in steel
p = ratio of longitudinal reinforcement
p' = ratio of spiral
n = ratio of moduli

The units used for loads, areas, and stresses are pounds, square inches, and pounds per square inch respectively.

TABLE 1. PROGRAM OF TESTS FOR SERIES 3

Design Strength of Concrete lb. per sq. in.			Reinforcement: Longitudinal per cent			Reinforcement: Spiral per cent			Storage		Method of Test		Number of Columns: Alike	Number of Columns: Total
2000	3500	5000		4		0	1.2	2	Wet	Dry	A. C. I. Load	No Load	2	72
2000	3500	5000	1½		6			2		Dry	A. C. I. Load	No Load	2	24
	3500		1½		6	0	1.2			Dry	A. C. I. Load		2	8
	3500		1½		6			2		Dry	New York Load		2	4
												Total		108

steel. Three percentages of longitudinal reinforcement, approximately 1½, 4, and 6 per cent respectively, were used. The spiral reinforcement amounted to 0 per cent, 1.2 per cent or 2 per cent of the core area of the column. Some of the columns are stored in the moist room, while the others are stored in the testing laboratory during the period of sustained load. Companion columns under no load are stored with the loaded columns.

The columns have an outside diameter of 8¼ in., a core diameter (outside diameter of spiral) of 8 in., and an overall length of 60 in. The longitudinal bars are 60 in. long and have milled ends, thus insuring direct bearing on both steel and concrete.

Six 6 by 12-in. control cylinders were made with each column, three to be tested when the working load was applied to the column, and three to be stored with the column and tested at the time of loading the column to failure. The working load was applied when the columns were 56 days old. This load is to be maintained for one year and then the columns are to be tested to failure. The first group of columns was placed under load on June 12, and the last on August 15, 1930.

The present schedule calls for deformation readings on the columns immediately before and after the application of the working load, then at 1 day, 1 week, 2 weeks, 4 weeks, 12 weeks, 20 weeks, 32 weeks, and 52 weeks, after the application of the load. Up to the present time this schedule has been adhered to. With each set of observations on the loaded columns, a set is taken on the companion columns stored under no load. This report contains data obtained during the first 20 weeks of the test.

3. *Acknowledgment.*—The success of the work has been promoted by the interested cooperation of the men connected with it. Especial mention is made of C. L. Kreidler, Research Fellow, and of C. C. Keyser, Laboratory Assistant, for their valuable assistance in carrying out the tests and preparing the data for publication.

4. *Proportioning and Strength of Concrete.*—The cement and aggregates used in this series came from the same supply as that used in Series 1 and 2. The mixes used for the different strengths

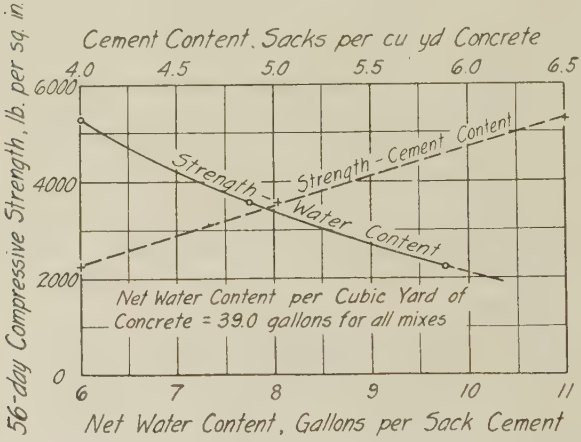


FIG. 1—RELATION BETWEEN STRENGTH AND WATER CONTENT AND BETWEEN STRENGTH AND CEMENT CONTENT FOR CONCRETE USED IN SERIES 3

of concrete were the same as those used in Series 2. Two batches of concrete were required for each column and its six control cylinders. Four minutes mixing was used, one minute dry, and three minutes wet. The two batches were dumped into a metal container and mixed together by hand before placing in the column mold. Table 2 shows the uniformity in strength and slump. The uniformity throughout the series is good, although nearly nine weeks elapsed between the making of the first and the last columns designed to have concrete of the same strength.

TABLE 2. DATA OF CONCRETE OF SERIES 3

Date of Make	Mix by Weight	Water-Cement Ratio by Weight	Slump in.	Strength at 56 Days lb. per sq. in.
Designed for Compressive Strength of 2000 Lb. per Sq. In.				
1930				
4–21	1 : 3.42 : 5.13	.864	3¼	2140
4–23	1 : 3.42 : 5.13	.864	2¾	2180
5– 5	1 : 3.42 : 5.13	.864	2¾	2240
5–19	1 : 3.42 : 5.13	.864	3	2210
5–21	1 : 3.42 : 5.13	.864	3	2240
6– 2	1 : 3.42 : 5.13	.864	2½	2270
6– 4	1 : 3.42 : 5.13	.864	3	2320
6–18	1 : 3.42 : 5.13	.864	2½	2290
Designed for Compressive Strength of 3500 Lb. per Sq. In.				
4–17	1 : 2.65 : 4.00	.686	4¼	3380
4–18	1 : 2.65 : 4.00	.686	4½	3330
4–25	1 : 2.65 : 4.00	.686	4½	3540
4–28	1 : 2.65 : 4.00	.686	4¼	3350
5– 8	1 : 2.65 : 4.00	.686	5¼	3560
5–12	1 : 2.65 : 4.00	.686	5	3730
5–16	1 : 2.65 : 4.00	.686	5¼	3780
5–23	1 : 2.65 : 4.00	.686	4¼	3700
5–26	1 : 2.65 : 4.00	.686	4½	3870
6– 6	1 : 2.65 : 4.00	.686	4¼	3900
6– 9	1 : 2.65 : 4.00	.686	4	3670
6–16	1 : 2.65 : 4.00	.686	3¾	3240*
Designed for Compressive Strength of 5000 Lb. per Sq. In.				
4–30	1 : 1.98 : 2.96	.531	6	5180
5– 2	1 : 1.98 : 2.96	.531	5	4600†
5–14	1 : 1.98 : 2.96	.531	5¾	5560
5–28	1 : 1.98 : 2.96	.531	4¾	5670
5–30	1 : 1.98 : 2.96	.531	4¾	5600
6–11	1 : 1.98 : 2.96	.531	5½	5120
6–13	1 : 1.98 : 2.96	.531	5½	5270
6–20	1 : 1.98 : 2.96	.531	4½	5050

*Eccentric Loaded. †Due to split head of testing machine.

Table 3 gives the summarized data of the concrete. A good agreement was found between the strength designed and that secured. The water content was 39.0 gal. per cu. yd. of concrete for all mixes. This produced a placeable concrete and only a small variation was found in the consistency as measured by the slump. Fig. 1 shows the relation between the strength of the concrete and the water-cement ratio used in the mix. The strength-water-content relation was a smooth curve very similar to that obtained for Series 2. The relation between cement content and strength is given by the straight, broken line. This diagram confirms the results of former tests which have indicated that with the same aggregates and a constant water content per cu. yd. of concrete, the slump was approximately constant and

TABLE 3. SUMMARIZED DATA OF CONCRETE OF SERIES 3

Strength of Concrete at 56 Days lb. per sq. in.		Mix by		Net Water Content* Gallons per		Cement Content Sacks pe. cu. yd. Concrete	Average Slump in.
Designed	Obtained	Weight	Absolute Volume	Sack Cement	Cu. Yd. Concrete		
2000	2230	1 : 3.42 : 5.13	1 : 4.08 : 6.11	9¾	39.0	4.00	2¾
3500	3590	1 : 2.65 : 4.00	1 : 3.16 : 4.77	7¾	39.0	5.02	4½
5000	5260	1 : 1.98 : 2.96	1 : 2.36 : 3.53	6	39.0	6.50	5¼

*Assumed 0.7 per cent absorption by weight of dry aggregates.

the strength increased in proportion to the increase in the cement content.

5. *Modulus of Elasticity of Concrete.*—The modulus of elasticity was determined on one control cylinder from each column. The apparatus used was the same as that described in the report of Series 2. The results obtained were averaged for each type of concrete used, and these averages are plotted in Fig. 2. The slope of the tangent at a stress of 500 lb. per sq. in. was taken as the modulus of elasticity of the concrete.

6. *Reinforcement.*—The longitudinal reinforcement consisted of intermediate grade deformed steel bars and came from the same supply as that described in Series 2. The bars were 22 ft. long when received. At the laboratory they were cut into four 5 ft. lengths and a 2-ft. test coupon. The coupons from each set of four bars were tested for yield-point stress and ultimate strength. The summary of these tests is given in Table 4. Both yield-point stress and the ultimate strength of the coupons (based upon nominal areas) varied considerably. The variation in weights of bars of the same nominal size indicates that the principal variation in strength was probably due to the variation of the cross-section of the bars, rather than to difference in quality of the steel.

The lateral reinforcement consisted of intermediate grade wire spirals. The spirals had an outside diameter of 8 in., and an overall length of 59 in. Three extra turns were provided at each end of the spiral for anchorage. A No. 5 rod (nominal diameter 0.207 in.) having a 1.35-in. pitch gave 1.2 per cent lateral reinforcement and a ¼-in. rod having a 1.20-in. pitch gave 2 per cent lateral reinforcement. An identified test coupon accompanied

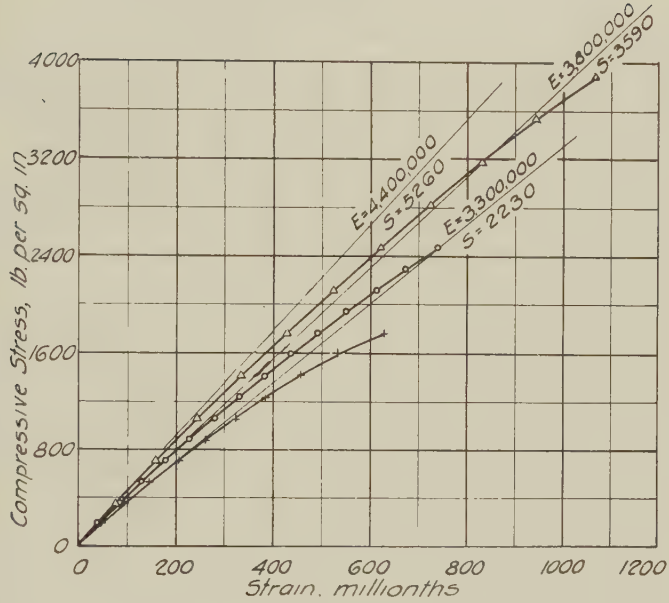


FIG. 2—AVERAGE STRESS-STRAIN DIAGRAM FOR CONTROL CYLINDERS OF SERIES 3

each spiral as received. The average tensile strengths of these coupons were 83,200 and 73,400 lb. per sq. in. for the No. 5 and the ½-in. rods respectively (see Table 5). It will be noted that the ultimate strength of the No. 5 rod was considerably above that of the ¼-in. rod and that of the longitudinal reinforcement.

As described in the report for Series 2, the reinforcement for columns having both longitudinal and spiral reinforcement was assembled into a unit before being placed in the mold.

In the columns having longitudinal reinforcement but no lateral reinforcement, the bars were held in position by tying to five No. 10 wire hoops equally spaced. These hoops did not provide appreciable lateral reinforcement, both because of the large spacing and because in each hoop the ends of the wires were merely hooked together.

7. *Placing and Curing of Concrete.*—The placing and curing of the concrete were the same as those used in Series 2. When

TABLE 4. SUMMARY OF TESTS OF LONGITUDINAL BARS

Nominal Size of Bar	Number of Coupons Tested	Yield Point Stress lb. per sq. in.	Ultimate* Strength lb. per sq. in.
½-in. square	144	43,000	66,480
½-in. round	18	49,490	74,490
⅝-in. round	18	43,780	72,380
¾-in. round	18	44,930	68,980

*Based on nominal areas.

TABLE 5. RESULTS OF TESTS OF SPIRAL COUPONS

Nominal Size of Bar	Number of Coupons Tested	Average Tensile Strength* lb. per sq. in.
No. 5 (0.207-in. diam.)	29	83,200
¼-in. diam.	60	73,400

*Based on nominal areas.

four columns of the same mix and reinforcement were concreted on the same day, two were for test under sustained working load. and the two companion columns were for storage under no load, Some of the columns tested under sustained load did not have companion columns stored under no load.

The columns and their control cylinders were cured in the moist room for 56 days.

8. *Working Loads.*—From the standpoint of interpreting the data it would have been desirable to use such a test load as would produce the same initial strain in all columns of the same concrete strength. It was not possible, however, to do this and at the same time to use as test loads the working loads computed from the building codes referred to. The amount of variation of the strain in the concrete at the application of the load is indicated in Table 6.

The formula given by the American Concrete Institute Code for spirally reinforced columns was used in determining the test loads for the columns without, as well as for those with spiral reinforcement. This could be done consistently because there is no term in the formula which represents the amount of spiral re-

TABLE 6. TAUBLATION OF STRAINS AND STRESSES

Column No.		Design Str. of Concrete, lb. per sq. in.	Reinf Per Cent		Total Load lb.	Conditions at Loading			Conditions at Twenty Weeks					
									Effect of Shrinkage and Flow			Effect of Shrinkage		
						Strain Millionths	Stress, Lb. per sq. in.		Strain Millionths	Stress, lb. per sq. in.		Strain Millionths	Stress, lb. per sq. in.	
Loaded	Not Loaded		Long.	Spiral			Concrete	Steel		Concrete	Steel		Concrete	Steel
										**	**		**	**
Wet Storage														
9, 10	11, 12	2,000	4	0	64,400	421	810	12,630	627	550	18,810	40	—50	1,200
17, 18	19, 20	3,500	4	0	79,500	384	1,170	11,520	576	930	17,280	40	—50	1,200
25, 26	27, 28	5,000	4	0	96,500	396	1,510	11,880	604	1,250	18,120	17	—20	510
53, 54	55, 56	2,000	4	1.2	64,400	354	890	10,620	524	680	15,720	88	—110	2,640
61, 62	63, 64	3,500	4	1.2	79,500	414	1,130	12,420	585	920	17,550	36	—40	1,080
69, 70	71, 72	5,000	4	1.2	96,500	414	1,480	12,420	634	1,210	19,020	32	—40	960
77, 78	79, 80	2.000	4	2	64,400	341	910	10,230	522	680	15,660	42	—50	1,260
85, 86	87, 88	3,500	4	2	79,500	358	1,200	10,740	542	970	16,260	18	—20	540
93, 94	95, 96	5,000	4	2	96,500	441	1,450	13,230	612	1,240	18,360	26	—30	780
Dry Storage														
1, 2		3,500	1½	0	49,000	298	850	8,940	1,206	420	36,180			
3, 4		3,500	6	0	109,000	488	1,380	14,640	1,086	250	32,580			
5, 6	7, 8	2,000	4	0	64,400	316	940	9,480	972	130	29,160	168	—210	5,040
13, 14	15, 16	3,500	4	0	79,500	369	1,190	11,070	1,170	190	35,100	241	—300	7,230
21, 22	23, 24	5,000	4	0	96,500	447	1,440	13,410	1,151	570	34,530	253	—310	7,590
29, 30	31, 32	2,000	1½	2	38,400	232	670	6,960	906	340	27,180	194	—90	5,820
33, 34		3,500	1½	1.2	49,000	312	840	9,360	1.053	490	31,590			
35 36		3,500	6	1.2	109,000	441	1,470	13,230	996	420	29,880			
37, 38	39, 40	3,500	1½	2	49,000	246	870	7,380	1,027	500	30,810	333	—160	9,990
41, 42	43, 44	5,000	1½	2	60,300	345	1,050	10,350	1,123	680	33,690	375	—180	11,250
45, 46		3,500	1½	2	87,500*	390	1,180	11,700	1,422	690	42,660			
47, 48		3,500	6	2	82,300*	370	1,040	11,100	959	—80	28,770			
49, 50	51, 52	2,000	4	1.2	64,400	342	910	10,260	974	120	29,220	212	—260	6,360
57, 58	59, 60	3,500	4	1.2	79,500	396	1,150	11,880	1,038	360	31,140	268	—330	8,040
65, 66	67, 68	5,000	4	1.2	96,500	403	1,520	12,090	1,049	690	31,470	249	—310	7,470
73, 74	75, 76	2,000	4	2	64,400	355	890	10,650	1,022	60	30,660	206	—260	6,180
81, 82	83, 84	3,500	4	2	79,500	380	1,170	11,400	1,036	360	31.080	225	—280	6,750
89, 90	91, 92	5,000	4	2	96,500	411	1,490	12,330	1,045	700	31,350	299	—370	8,970
97, 98	99, 100	3,500	6	2	109,000	490	1,380	14,700	1,048	320	31,440	195	—370	5,850
101, 102	103, 104	2,000	6	2	90,500	485	990	14,550	982	50	29,460	144	—270	4,320
105, 106	107, 108	5,000	6	2	130,600	535	1,750	16,050	1,125	630	33,750	239	—450	7,170

*Load according to New York Building Code, remainder according to A.C.I. Building Code
**No sign indicates compression, minus sign indicates tension.

inforcement, and it was desirable to do it because it permitted a direct comparison of the deformations under the same load for columns without as for those with spiral reinforcement.

9. *Applying Working Load.*—At the age of 56 days the columns were removed from the moist room. The design load was applied to the columns scheduled to be held under load for one year. The two columns of the same mix and the same reinforcement were placed one on top of the other so that one loading rig served for

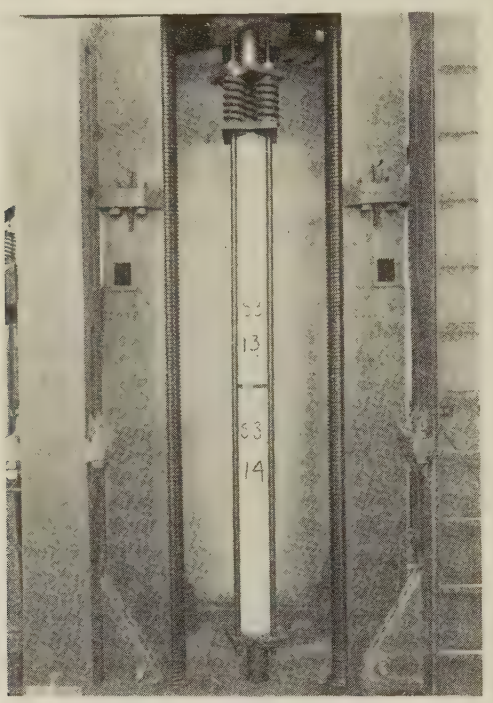


FIG. 3—APPLYING WORKING LOAD TO COLUMNS

both columns. The loading rig and the method used in applying the load to two columns simultaneously are shown in Fig. 3. The load was applied through helical springs held between two loading plates at the upper end of the rig. Steel rods, 1½ in. in diameter and 12½ ft. long, transmitted the load from the springs to the columns. In order to secure a good distribution of the load, planed steel bearing plates were used at the ends of the columns and pieces of beaver board were inserted between the bearing plates and the loading plates. A steel plate, planed on both faces, was inserted between the columns.

The rods and loading plates were donated by the Bethlehem Steel Co. of Bethlehem, Pennsylvania.

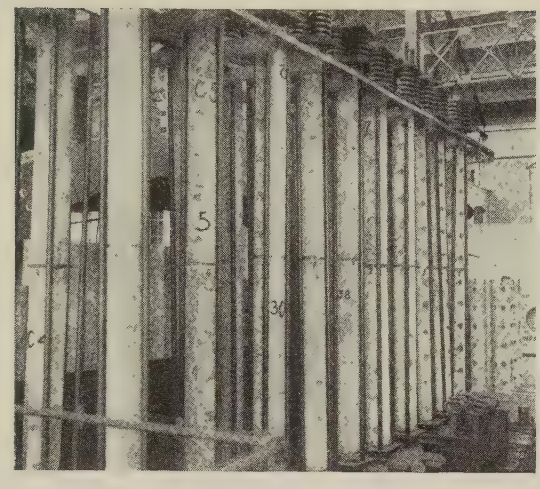


FIG. 4—COLUMNS OF SERIES 3 UNDER SUSTAINED LOAD IN DRY STORAGE

The load was applied to the columns in an 800,000-lb. vertical-screw testing machine. The two columns and the loading rig were assembled on the table of the machine and the design load applied by bringing the head of the machine down upon the upper plate, which bore upon the springs. The length of the springs in their compressed condition under the design load was measured to the nearest 0.001 in. A small excess load was then applied to compensate for the elongation of the rods and deflection of the plates to be expected upon the raising of the head of the testing machine. The nuts were then tightened on the rods, after which all the load exerted by the testing machine was released. The length of the springs was then remeasured, and if it did not agree with that which had been found under the design load, successive trials were made until agreement was secured.

Strain gage readings were taken on all columns immediately before and after the application of the load. The loaded columns and their unloaded companion columns were then stored as scheduled, either wet or dry. The columns stored dry are shown in Fig. 4.

10. *Maintaining the Working Load on Columns in Dry Storage.* —The springs are holding the load fairly well. However, there

is a slight falling off of load, due to the deforming of the columns, and also to permanent set in the springs. The length of the springs was measured after each series of strain gage readings. After 12 weeks under load it appeared that the falling off of the load was of sufficient importance to necessitate readjustment of the springs. The loading rig containing the columns was placed in the testing machine and the correct working load re-applied. The method used for applying the load was also used for the readjustment. The average decrease in 12 weeks was about 7 per cent of the correct working load. Of this, about 4 per cent was due to the deformation of the columns and 3 per cent due to permanent set in the springs.

The columns stored in the moist room showed so slight an increase in deformation during the time under load that it has been unnecessary to readjust the loads.

11. *Data Obtained During the First Twenty Weeks Under Load.*—For purposes of reference the average measured unit deformations for each pair of companion columns of Series 3, are given at the end of this report in Figs. 15 to 44. The agreement between companion columns was very good as is illustrated in Fig. 5. It is not feasible in this report, to include similar results for all columns.

There were 20 gage lines on each column for the measurement of deformation in the steel and 20 for the concrete. Two readings were taken on each gage line with each observation. With some exceptions, the results of 80 readings (40 on each of two columns) therefore were averaged to give each plotted point in Figs. 15 to 44.

The agreement between the deformations observed in the concrete and those in the steel was very good. The most important exception noted is that of columns 81 and 82 (Fig. 38) which show a difference in deformation of about 50 millionths.

The average of longitudinal strains measured at opposite ends of any diameter has been found equal (or nearly so) to the average of the longitudinal strains at opposite ends of the diameter lying at right angles to, and in the same horizontal plane with the former diameter. This applied not only to the elastic strains

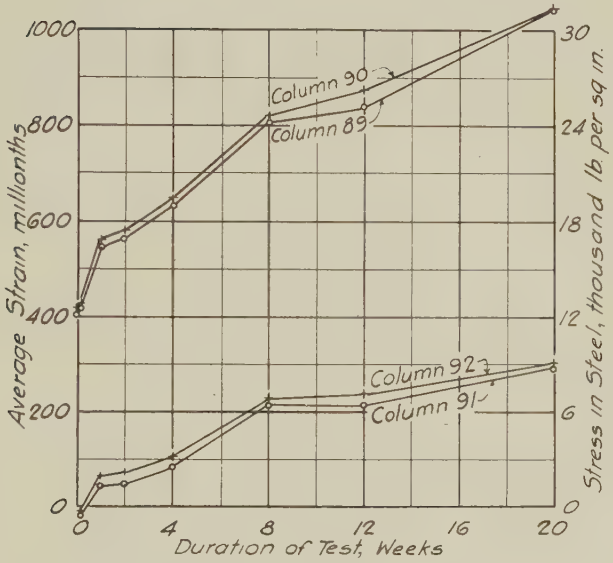


FIG. 5—AGREEMENT BETWEEN DEFORMATION OF COMPANION COLUMNS

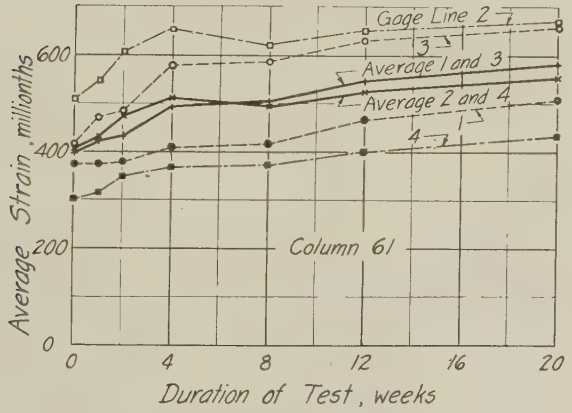


FIG. 6—ILLUSTRATION OF CONSERVATION OF PLANE SECTION

measured at the time of application of load, but also to the shrinkage and increase in deformation during an extended period. An illustration of the relations here pointed out is given in Fig. 6. In the working up of the strain data reliance was placed upon this relation for the detection of error in the computations.

All the columns in dry storage generally showed large increase in deformation during the first two to four weeks under sustained load. From then on the rate of increase in deformation became smaller and more nearly constant as time progressed. The companion columns under no load showed a steady shrinkage for the first 12 weeks of test, but from then on there was little increase in deformation due to shrinkage. The shrinkage deformations of the unloaded columns were subtracted from the deformations of the companion columns under load when both were present. The resulting time-deformation curves so corrected are shown in Fig. 15 to 44 as broken lines. Each corrected curve follows approximately a straight line from about two weeks after the application of the load until the end of the twenty weeks period.

The columns in wet storage showed a much smaller increase in deformation during the loading period than did those in dry storage. The deformation of the unloaded companion columns was quite small throughout the test. As with the columns stored dry each corrected deformation curve for the loaded columns stored wet followed approximately a straight line from about two weeks after the beginning until the end of the twenty weeks under load. It is noted that the increase in deformation due to the load was much smaller for the columns in wet storage than for those in dry storage.

12. *Effect of Variation in Strength of Concrete on Deformation.*—Figs. 7 and 8 give the unit deformations under load for columns made with concrete designed to have strengths of 2000, 3500, and 5000 lb. per sq. in. In Fig. 7 the solid lines and the broken lines represent the deformations of columns having 6 per cent and 1½ per cent longitudinal reinforcement. The spiral reinforcement was 2 per cent in all columns represented in Fig. 7. Each curve in Fig. 8 represents the measured strains for the columns with concrete of a certain designed strength. All the columns rep-

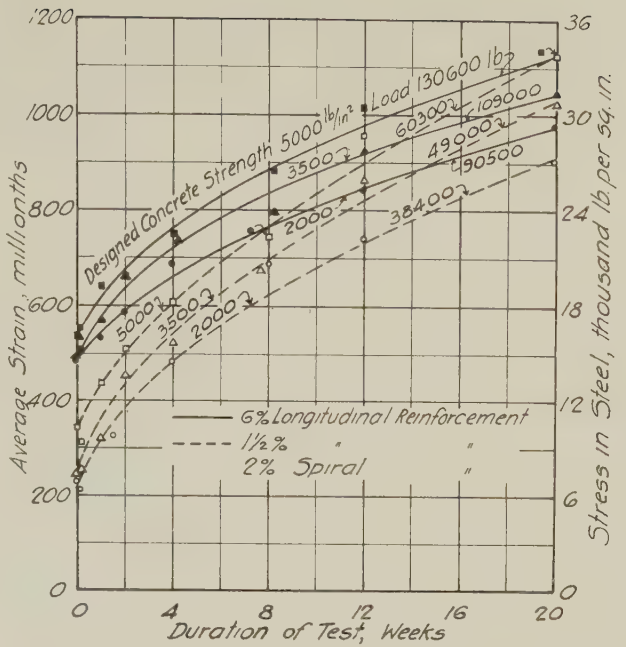


FIG. 7—EFFECT OF VARIATION IN STRENGTH OF CONCRETE ON INCREASE IN DEFORMATION OF COLUMNS HAVING 2 PER CENT SPIRAL REINFORCEMENT AND 1½ PER CENT AND 6 PER CENT LONGITUDINAL REINFORCEMENT, DRY STORAGE

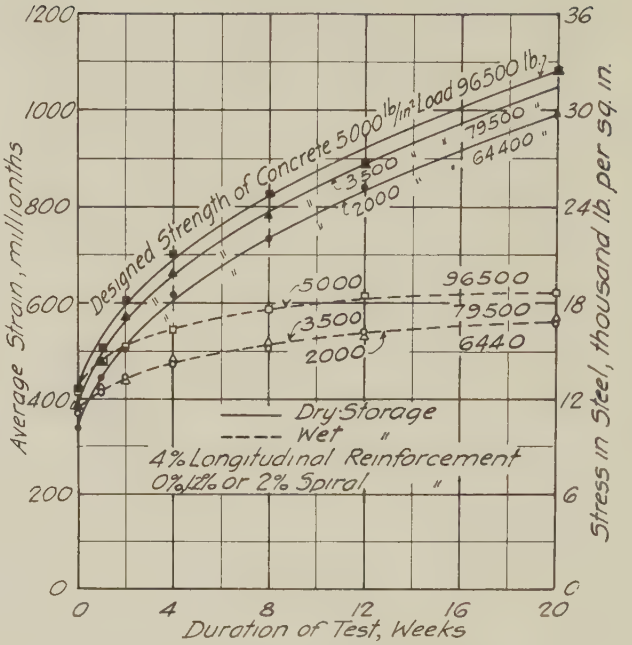


FIG. 8—EFFECT OF VARIATION IN STRENGTH OF CONCRETE ON INCREASE IN DEFORMATION OF COLUMNS HAVING 4 PER CENT LONGITUDINAL REINFORCEMENT AND 0 PER CENT, 1.2 PER CENT, OR 2 PER CENT SPIRAL REINFORCEMENT

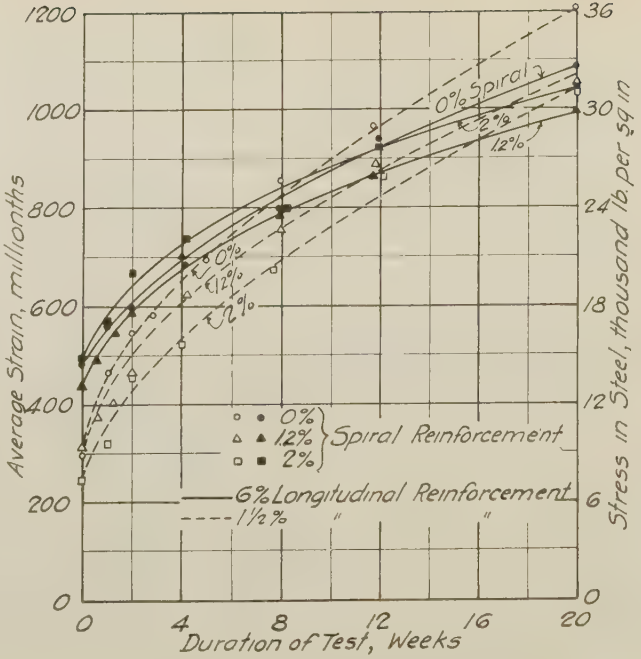


FIG. 9—EFFECT OF VARIATION IN PERCENTAGE OF SPIRAL REINFORCEMENT ON INCREASE IN DEFORMATION OF COLUMNS. DESIGNED STRENGTH OF CONCRETE 3500 LB. PER SQ. IN., 1½ PER CENT AND 6 PER CENT LONGITUDINAL REINFORCEMENT, DRY STORAGE

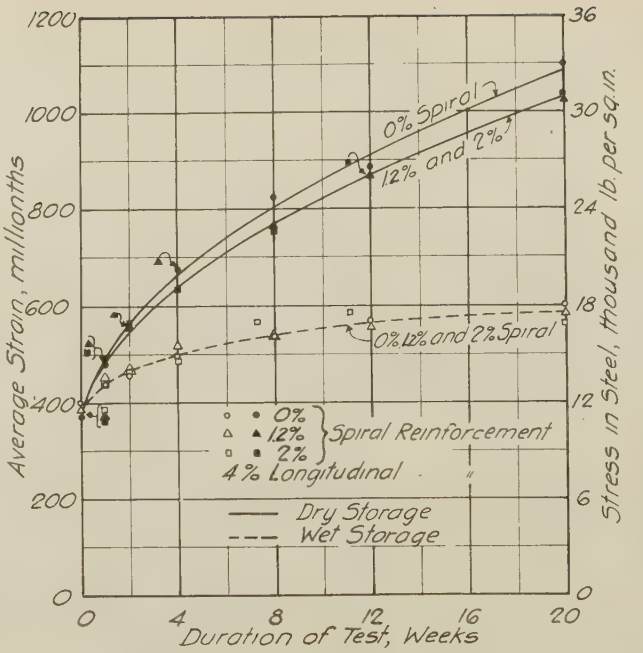


FIG. 10—EFFECT OF VARIATION IN PERCENTAGE OF SPIRAL REINFORCEMENT ON INCREASE IN DEFORMATION OF COLUMNS HAVING 4 PER CENT LONGITUDINAL REINFORCEMENT, DESIGNED STRENGTH OF CONCRETE 2000, 3500, AND 5000 LB. PER SQ. IN.

resented had 4 per cent of longitudinal reinforcement. The spiral reinforcement was either 0, 1.2, or 2 per cent. It is later shown that the strains were practically independent of the amount of the spiral reinforcement, and this fact is used as the justification for averaging the strains indiscriminately regardless of the amount of the spiral reinforcement. The solid lines represent the columns in dry storage and the broken lines those in wet storage. The most striking feature of these figures is that the strain curves for a given type of storage are substantially parallel. That is, within the range of these tests the rate of increase in deformation was everywhere practically independent of the strength of the concrete and of the initial stress. This comment applies to the columns held under load in wet storage as well as to those in dry storage. However, the amount of increase in deformation was very much smaller for the columns loaded in wet, than for those loaded in dry storage. It is probable that the reason for the apparent lack of dependence of the rate of increase in deformation upon the strength of the concrete is the fact that under the code used for determining the working load the concretes of the higher strengths required correspondingly greater loads. In other words, the effect of the higher strengths apparently was offset by the corresponding higher stresses.

13. *Effect of Amount of Spiral Reinforcement on Deformation.*— Fig. 9 and 10 give the time-deformation curves for columns with 0, 1.2, and 2 per cent spiral reinforcement. For all of the columns with a given percentage of longitudinal reinforcement and of a given concrete strength the same load was applied, regardless of the percentage of spiral used. At the time of applying the load the deformations were greatest in some instances for the columns having no spiral and in others for those having 1.2 or 2 per cent of spiral. This indicates that slight variations in the modulus of elasticity of the concrete had more effect on the amount of initial deformation than did the presence or absence of spiral reinforcement. In Fig. 9 the deformations of columns made of concrete having a designed strength of 3500 lb. per sq. in. are shown for 0, 1.2, and 2 per cent of spiral reinforcement and for 1½ and 6 per cent of longitudinal reinforcement. The solid lines represent the columns having 6 per cent and the broken lines those having 1½

per cent longitudinal reinforcement. All columns in this figure were stored dry. Fig. 10 shows the deformations obtained on columns having 4 per cent of longitudinal reinforcement. In Fig. 10 each point represents the average of the deformations of the columns having a given percentage of spiral reinforcement, regardless of the strength of the concrete, which varied between 2000, 3500, and 5000 lb. per sq. in. Since the same variations in strength of concrete enter for one curve as for another in this figure, the curves may be compared in studying the effect of the amount of spiral reinforcement on the deformations. The solid lines represent the deformations of the columns stored dry, and the broken lines those of the columns stored wet.

Within the range of accuracy of the results, except for the columns without spiral reinforcement, the time-deformation curves were either identical or parallel, depending on whether the deformations immediately after applying the load were equal or not. The results indicate therefore, that the rate of increase in deformation was practically independent of the percentage of spiral reinforcement.

14. *Effect of Variation in Longitudinal Reinforcement on Deformation.*—In Fig. 11 are shown the deformations which occurred in the columns during the first 20 weeks of test. To eliminate the irregularities caused by the small number of specimens, an average curve was prepared for all the columns having 1½ per cent longitudinal reinforcement, regardless of the percentage of the spiral, or the strength of the concrete. Similar average curves for 4 per cent and 6 per cent longitudinal reinforcement were prepared and all three are shown in Fig. 11.

The greater the percentage of longitudinal reinforcement, the greater were the applied loads and the initial deformation. This is because under the American Concrete Institute Code (which was used for determining the test loads) an increase in longitudinal reinforcement calls for an increase in the working stress in the concrete. The figure indicates that the rate of increase in deformation with the time under load was greatest for the smallest percentage of longitudinal reinforcement and the least for the largest percentage.

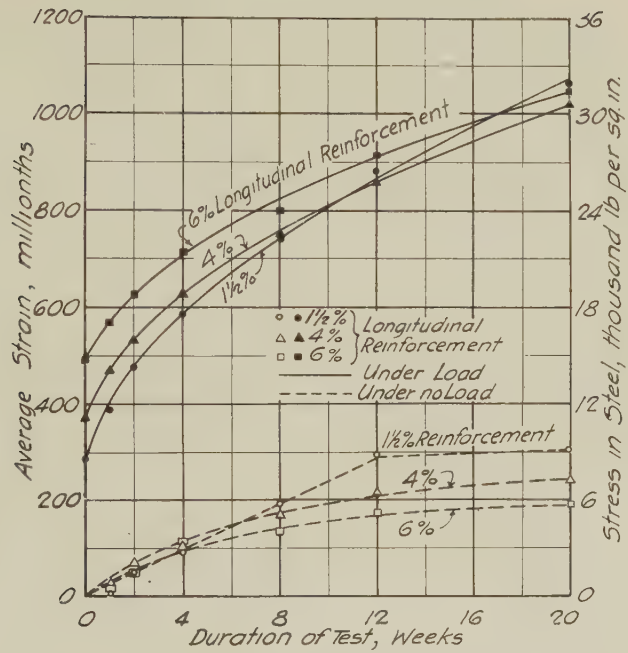


FIG. 11—AVERAGE DEFORMATION OF COLUMNS HAVING VARIOUS PERCENTAGES OF LONGITUDINAL REINFORCEMENT, DRY STORAGE

The broken lines in the lower part of Fig. 11 show the effect of the variation in percentage of longitudinal reinforcement upon the shrinkage of the columns under no load. For the first four weeks of the test, the shrinkage was nearly the same for the three different percentages of reinforcement. From then on, the shrinkage of the columns with the largest percentage of reinforcement was much smaller than the shrinkage of the columns having smaller percentages. After 20 weeks the shrinkage deformations of the columns having 6, 4, and 1½ per cent longitudinal reinforcement were 190, 240, and 300 millionths, respectively. If the shrinkage were subtracted from the companion columns under sustained load, the time-deformation curves would become nearly parallel except for slightly smaller vertical distance between the curves at the end of 20 weeks than at the beginning of the test. Evidently the resistance offered by the added reinforce-

ment offset the effect of the greater initial concrete stress on the rate of deformation for columns having the greater percentages of longitudinal reinforcement.

15. *Comparison Between New York and American Concrete Institute Building Codes.*—Fig. 12 shows the deformations of the two groups of companion columns loaded in conformity with the New York and the American Concrete Institute Building Codes. The solid lines represent the deformations of the columns loaded according to the New York Code, and the broken lines those loaded according to the Institute Code. It is noted that for the former the initial concrete stresses for the columns having 6 and 1½ per cent longitudinal reinforcement respectively, were practically equal. The rate of increase in deformation was much greater for the columns having 1½ per cent reinforcement than for those having 6 per cent. That is, the deformation curves departed continuously from each other as time progressed. At the end of 20 weeks the stress in the steel of the columns having 1½ per cent longitudinal reinforcement was about 14,000 lb. per sq. in. (48 per cent) greater than that for the columns having 6 per cent reinforcement.

The broken lines represent the deformations of the columns loaded according to the Institute Code. Here the curves approached each other as time progressed, and at the end of 20 weeks the deformations of the columns having 1½ and 6 per cent longitudinal reinforcement were nearly equal, even though the columns with 6 per cent had about twice as large initial deformation as did those with 1½ per cent reinforcement.

If, as is present practice, the stress in the longitudinal reinforcement is an important criterion in the design of concrete columns, the American Concrete Institute Building Code gave a better basis for designing reinforced concrete columns than did the present New York Building Code. There are, however, indications that the strength of a reinforced column is equal to the sum of the strength of the concrete and the yield-point strength of the longitudinal reinforcement. If this were to be established as the criterion for the design of columns, the importance of maintaining a fixed stress in the longitudinal reinforcement, regardless of its percentage, would become of less significance.

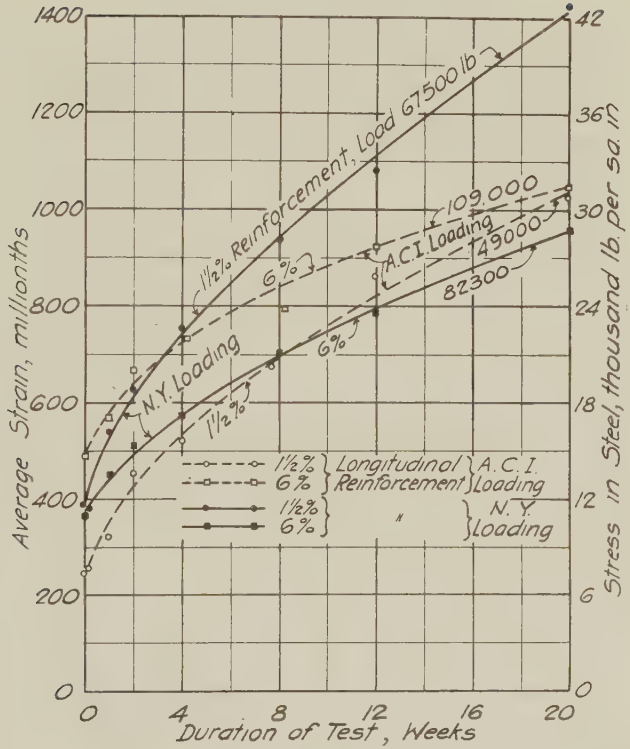


FIG. 12—DEFORMATION OF COLUMNS LOADED IN CONFORMANCE WITH THE NEW YORK AND THE AMERICAN CONCRETE INSTITUTE BUILDING CODES, DESIGNED STRENGTH OF CONCRETE 3500 LB. PER SQ. IN., 2 PER CENT SPIRAL REINFORCEMENT, DRY STORAGE

16. *Shrinkage of Columns Stored Dry.*—Fig. 13 shows the shrinkage for columns stored in the air of the laboratory. The shrinkage of columns having 1½, 4, and 6 per cent longitudinal reinforcement respectively, has been shown for each strength of concrete. Each point represents the average for all gage lines on all columns having concrete of the same strength and the same percentage of longitudinal reinforcement, regardless of the percentage of spiral reinforcement. It is noted that up to four weeks dry storage no marked difference was found for columns of different percentages of longitudinal reinforcement. From then on, however, the shrinkage for the columns having 1½ per cent

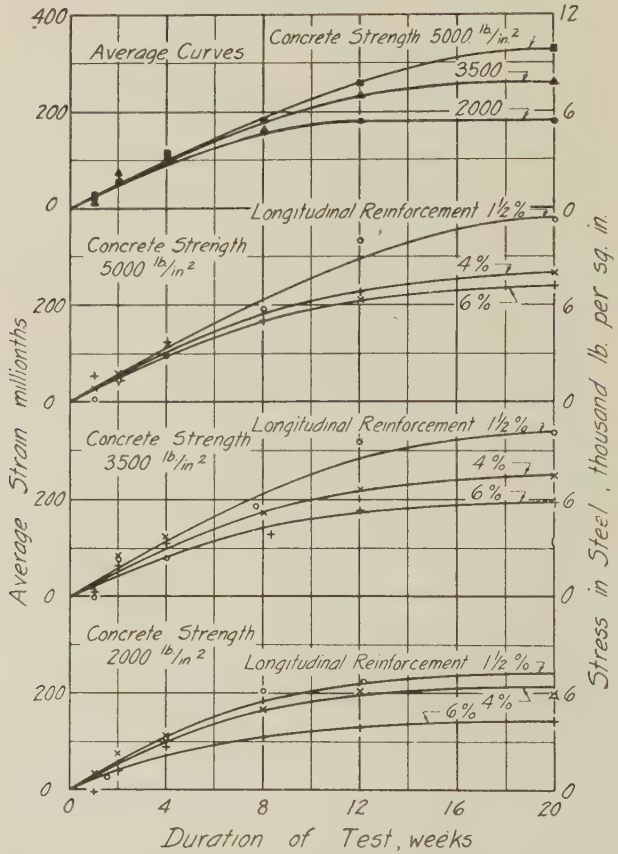


FIG. 13—SHRINKAGE OF COLUMNS WITH VARIOUS PERCENTAGES OF LONGITUDINAL REINFORCEMENT AND OF DIFFERENT STRENGTHS OF CONCRETE

was greater than that for columns having 4 per cent, and the latter in turn was greater than the shrinkage for columns having 6 per cent longitudinal reinforcement. With columns having a given percentage of longitudinal reinforcement, the greater the strength of the concrete the greater was the shrinkage. The effect of the strength of the concrete upon the shrinkage is shown

most directly in the top part of the diagram. Here the average shrinkage for each strength of concrete is shown, regardless of the percentage of longitudinal reinforcement. It was found that after the columns had been under test for more than four weeks the shrinkage was consistently greater with the concretes of the greater strengths.

17. *Summary of Deformations and Stresses*—Table 6 summarizes the deformations and the resulting stresses at the time of application of load and after 20 weeks of test, for the columns included in this series. The table includes the data of both sets of tests, those in which the columns had been loaded with the design load for 20 weeks, and those in which the columns were not loaded but in which the strains due to shrinkage had been measured. It includes also both sets of storage conditions, wet at a temperature of 70 deg. F., and dry at room temperature, which varied between 60 deg. F. and 95 deg. F. during the 20 weeks.

An illustration of how the stress in the steel increased and that in the concrete decreased during the time under sustained load is given in Fig. 14. The curves in the lower half of the diagram represent columns having a concrete strength of 5000, and those in the upper half of 2000 lb. per sq. in. In all these columns the reinforcement consisted of 2 per cent spiral and 1½ or 6 per cent longitudinal. The solid lines represent the stresses in the steel and the broken lines those in the concrete.

It is seen from Table 6 and Fig. 14 that the decrease in stress in the concrete due to the increase in deformation, was greater for the columns with high than for those with low percentages of longitudinal reinforcement. In the cases examined, this decrease was greater proportionately, as well as numerically. For columns 47 and 48 the deformations after 20 weeks of test indicated that a tension of 90 lb. per sq. in. was present in the concrete, that is, apparently the load carried by the reinforcement exceeded the total load on the columns.

Fig. 14 and Table 6 show that the increase in the stress in the steel was large and that it was greatest for the columns with the smallest percentage of longitudinal reinforcement. The highest

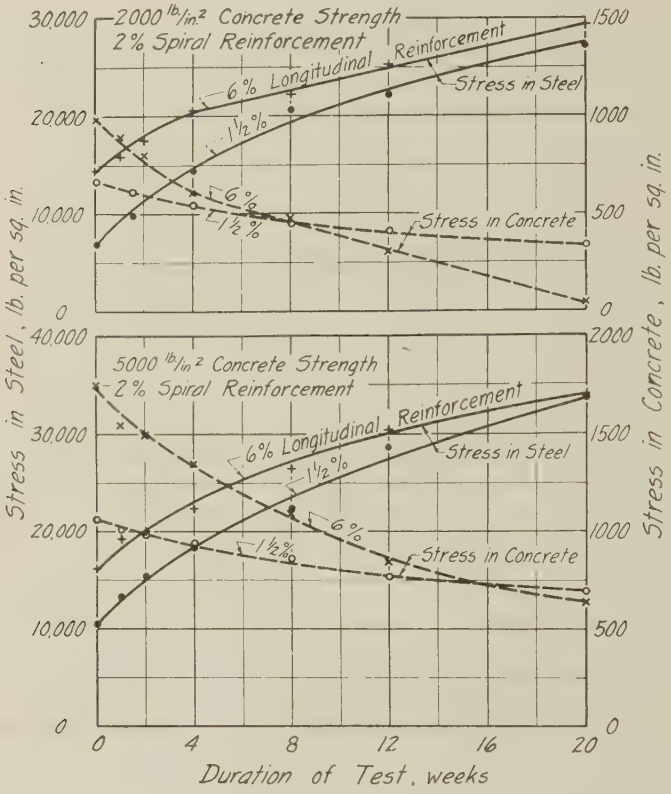


FIG. 14—EFFECT OF DURATION OF TEST ON STRESSES IN STEEL AND CONCRETE

stress reported in this table, 42,660 lb. per sq. in., is practically four times the initial stress. This stress is the average for columns 45 and 46 which have 1½ per cent longitudinal reinforcement. This shows that the steel stress after 20 weeks under the working load was not far from its yield-point stress of 49,500 lb. per sq. in.

For the columns not loaded the shrinkage caused a compression in the steel which resulted in a corresponding tension in the concrete, in spite of the fact that the concrete shortened. The total compressive force found as the product of the measured strain, the modulus of elasticity, and the total area of the steel, must have produced an equal total tensile force in the concrete, since

no external load was applied to the columns. The tensile stresses in the concrete, determined from this relation, are given in Table 6. It will be noted that the average tensile stress in the concrete of columns 107 and 108 was 450 lb. per sq. in. The next lower tensile stress was 370 lb. per sq. in., the average for columns 99 and 100. All four of these columns had 6 per cent longitudinal reinforcement.

A careful examination of all the columns which were stored under no load has recently been made and no cracks have been found. Especial attention was given to the search for cracks on those columns showing the highest tensile stresses. There is no probability that cracks were missed in this visual examination, for their presence would have been plainly indicated in the results of the strain gage measurements which included the total length of the column except for five inches at each end.

18. *Computation of Shrinkage and Flow.*—W. H. Glanville of the Department of Industrial Research, England,[5] has carried out researches on shrinkage and flow in concrete in which tests have been made and theoretical treatments given. The analysis of the problems is so fundamental, and the agreements between the results of the tests and the theoretical treatments are so marked as to make it worth while to include the lines along which the theoretical analysis was carried out. With minor modifications for notations and form of statement, the following are his theoretical analyses of the shrinkage and the flow of reinforced concrete members.

SHRINKAGE

If no slip occurs between the concrete and the steel, the actual change in length of the concrete in a reinforced concrete member is less than the shrinkage of the plain concrete member by an amount equal to the elastic tensile strain of the concrete due to the compressive stress induced in the steel plus the flow of the concrete which has taken place under its tensile stress. Since the increase in load in the steel has to be equal to the decrease in load in the concrete the following equation holds:

$$\Delta f_s \cdot a_s = \Delta f_c \cdot a_c$$

[5]Building Research Technical Papers "Studies in Reinforced Concrete," Technical Paper No. 11, "Shrinkage Stresses," and No. 12, "The Creep or Flow of Concrete Under Load."

where Δf_s and Δf_c are changes in stress in steel and concrete respectively, and a_s and a_c are the areas of steel and concrete respectively.

If c is the flow per unit stress up to the time under consideration the flow occurring during an increment of time Δt is $f_c \cdot \Delta c$, and, since the shrinkage of plain concrete less the flow must be equal to the sum of the strains in the steel and in the concrete, the following equation is derived:

$$\Delta s - f_c \cdot \Delta c = \frac{\Delta f_s}{E_s} + \frac{\Delta f_c}{E_c}$$

$$\text{Since } \Delta f_s \cdot a_s = \Delta f_c \cdot a_c$$

$$\Delta f_s = \frac{a_c}{a_s} \cdot \Delta f_c$$

$$\text{and } \Delta s - f_c \cdot \Delta c = \frac{\Delta f_c}{E_s} \cdot \frac{a_c}{a_s} + \frac{\Delta f_c}{E_c} = \Delta f_c \left(\frac{1}{E_s} \cdot \frac{a_c}{a_s} + \frac{1}{E_c} \right)$$

$$\text{or writing } \alpha = \frac{1}{E_s} \cdot \frac{a_c}{a_s} + \frac{1}{E_c}$$

we have in the limit:

$$ds - f_c \cdot dc = \alpha \cdot df_c$$

This relation can be solved if the relations between shrinkage and time and between flow and time are known. If the flow is neglected in the relation between the shrinkage and induced concrete stresses the formula becomes, by integration,

$$s = f_c \left(\frac{1}{E_s} \cdot \frac{a_c}{a_s} + \frac{1}{E_c} \right) = f_s \left(\frac{1}{E_c} \cdot \frac{a_s}{a_c} + \frac{1}{E_s} \right)$$

Mr. Glanville suggests that this formula be used but that the value of E_c be replaced by fictitious or "effective" modulus of elasticity E'_c in such a manner that the effect of the flow is allowed for.

FLOW

If a reinforced concrete member is under a constant load, the flow of the concrete during a small increment of time, Δt, under a unit stress is Δc. The modulus of elasticity of the concrete is assumed to be constant. Since the total flow is equal to the sum of the resulting strains in the concrete and that in the steel the following equation is derived:

$$f_c \cdot \Delta c = \frac{\Delta f_s}{E_s} - \frac{\Delta f_c}{E_c}$$

The increase in load in the steel must be equal to the decrease in the load in the concrete:

$$\Delta f_s \cdot a_s = -\Delta f_c \cdot a_c, \text{ or } \Delta f_s = -\frac{a_c}{a_s} \cdot \Delta f_c$$

substituting this value for Δf_s in the previous equation gives:

$$f_c \cdot \Delta c = -\frac{a_c}{a_s} \cdot \frac{\Delta f_c}{E_s} - \frac{\Delta f_c}{E_c} = -\Delta f_c \left(\frac{a_c}{a_s} \cdot \frac{1}{E_s} + \frac{1}{E_c}\right)$$

$$\text{or} \quad \Delta f_c = -\frac{f_c \cdot \Delta c}{\frac{a_c}{a_s} \cdot \frac{1}{E_s} + \frac{1}{E_c}}$$

$$\text{As before} \quad \alpha = \frac{a_c}{a_s} \cdot \frac{1}{E_s} + \frac{1}{E_c}$$

$$\text{Then} \quad \Delta f_c = -\frac{f_c \cdot \Delta c}{\alpha} \quad \text{or} \quad \frac{\Delta f_c}{f_c} = -\frac{\Delta c}{\alpha}$$

In the limit:

$$\frac{df_c}{f_c} = -\frac{d_c}{\alpha}$$

When $c = 0, f_c = f_o$ where f_o = stress in concrete immediately after loading. This expression may be integrated between the limits f_o and f_c and the result is:

$$f_c = \frac{f_o}{e^{c/\alpha}}$$

where c is the flow of plain concrete under unit stress from the time of loading to the time under consideration, and e is the base of Naperian logarithms.

The stresses in the steel and in the concrete are found from the following relations:

$$f_s = f_a + f'_s$$

$$f'_c = f_o - f_c = f_o \left(1 - \frac{1}{e^{c/\alpha}}\right)$$

$$\text{and} \quad f'_s = f_a \cdot \frac{a_c}{a_s} \cdot \frac{1}{n} \cdot \left(1 - \frac{1}{e^{c/a}}\right)$$

where:

f_s and f_c are the actual stresses in steel and concrete at the time under consideration (disregarding shrinkage effect), f_a and f_o are the stresses in steel and concrete at the time of application of load, f'_s and f'_c are the changes in stress in steel and concrete due to flow, and n is the ratio, $\frac{E_s}{E_c}$, between moduli.

As shrinkage and flow act independently of each other, the combined result is obtained by adding them.

19. *Review of the Problem.*—The findings reported herewith confirm in a striking manner the results of studies made as long ago as 1916. In that year A. H. Fuller and C. C. More presented a paper dealing with "Time Tests of Concrete." At the same meeting E. B. Smith of the Bureau of Public Roads gave a paper on "The Flow of Concrete Under Sustained Loads." Both papers presented evidence of the flow of concrete under load but did not show the strains in reinforcement embedded in the concrete. F. R. McMillan, however, discussed both papers and described results of measurements which he had initiated on reinforced concrete columns in buildings in service. The observations were begun about two months after the placing of the columns and had been in progress about seven months at the time of the first report. It was already evident that considerable shortening of the concrete and steel was taking place. In the following year (1917) Smith presented to the American Concrete Institute a paper embodying the results of a continuation of his studies. McMillan discussed this paper also, showing that the strains in the columns under measurement had reached a value corresponding, in one instance (designated as column 19), to a stress in the steel as high as 36,000 lb. per sq. in. In 1921 M. B. Lagaard presented as discussion of McMillan's paper "A Study of Column Test Data" further results of the studies initiated by McMillan on columns in service. By that time the measurements had been carried on for approximately six years. The strains had increased so that the apparent stress in the steel of column 19 was 45,000 lb. per sq. in. It is not unlikely that even

then the yield-point stress had been exceeded, especially in view of the fact that the measurements were not started until the columns were two months old. As no comment was made on the appearance of the column it is probable that there were no indications of distress.

R. E. Davis reports that in his investigation of flow the strength of plain concrete was uninfluenced by having been subjected to a long-time load of 800 lb. per sq. in. which caused a final deformation of 900 millionths or three times that with which the test started. Other of his tests confirm these results. If, therefore, the flow in a reinforced concrete column continues to the point at which the steel is stressed beyond the yield point, it is reasonable to expect that the concrete is still capable of carrying its full share of the load.

The only occasion for concern then, must be as to whether the carrying capacity of the steel is reduced by being stressed beyond the yield point. The tests have not yet been carried far enough to give the answer from the experimental end, but the question may reasonably be asked: If the longitudinal steel receives from the spiral and from its embedment in the concrete, sufficient lateral support to prevent buckling under the yield-point stress, how can it escape from continuing to carry its full share of the load?

At the risk of anticipating too far what the final outcome will be, it seems imperative that enough speculation be indulged in to enable us to visualize what must yet be done before the reinforced concrete column problem is solved. First, the strength of the column appears to depend upon the sum of the strengths of the concrete and the longitudinal reinforcement regardless of the ratio of the moduli of elasticity. Second, it seems evident that the basis for the design of reinforced concrete columns must be changed accordingly.

The tests must be carried far enough to determine what is the highest axial load that a reinforced concrete column will carry indefinitely without failure.

Tests must also be carried out on columns subjected to combined bending and direct stress. These tests must be made in such a way that some generalization may be made as to what

redistribution of the bending moment initially present in an indeterminate structure takes place as the concrete shrinks, and flows under long time load.

Readers are referred to the JOURNAL *for June, 1931, for discussion of the Reinforced Concrete Column Investigation of Committee 105. Such discussion should reach the Secretary by May 1, 1931.*

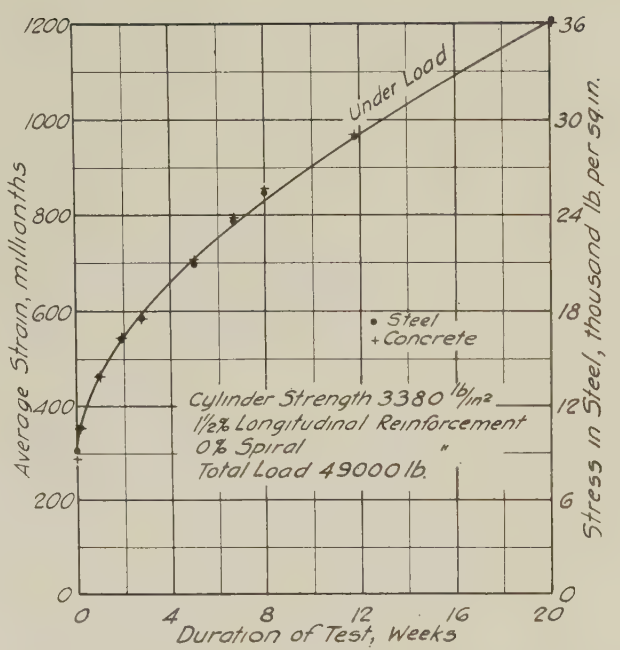


FIG. 15—DEFORMATION OF COLUMNS 1 AND 2 UNDER SUSTAINED LOAD IN DRY STORAGE

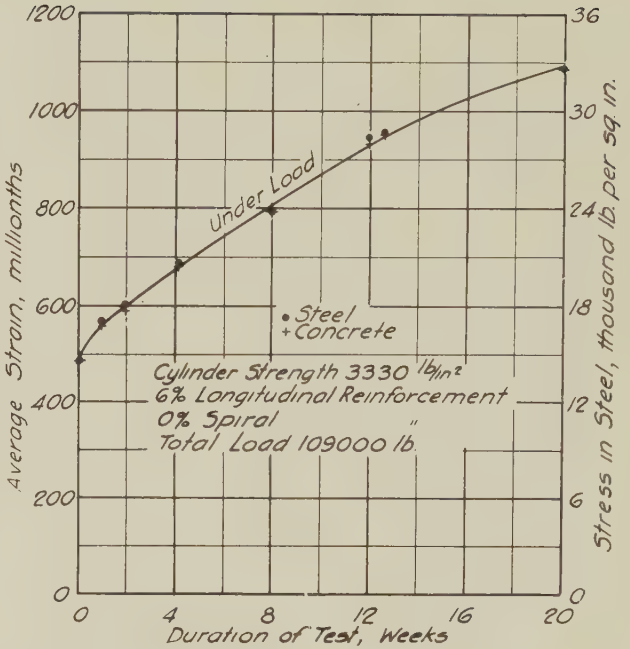


FIG. 16—DEFORMATION OF COLUMNS 3 AND 4 UNDER SUSTAINED LOAD IN DRY STORAGE

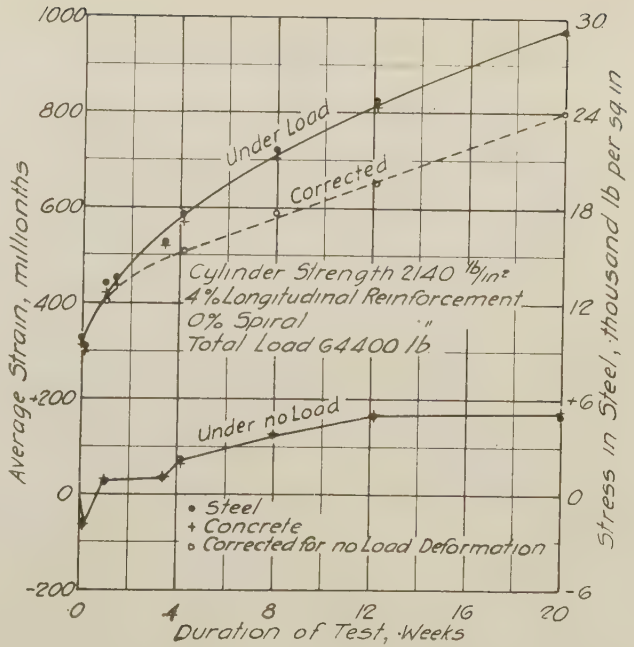


FIG. 17—DEFORMATION OF COLUMNS 5 AND 6 UNDER SUSTAINED LOAD AND OF COLUMNS 7 AND 8 UNDER NO LOAD IN DRY STORAGE

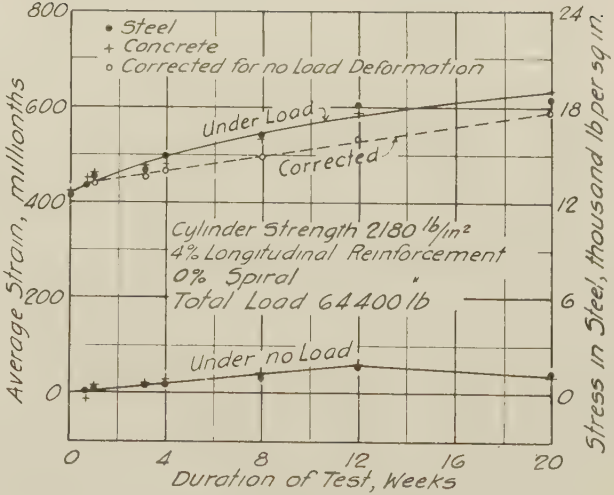


FIG. 18—DEFORMATION OF COLUMNS 9 AND 10 UNDER SUSTAINED LOAD AND OF COLUMNS 11 AND 12 UNDER NO LOAD IN WET STORAGE

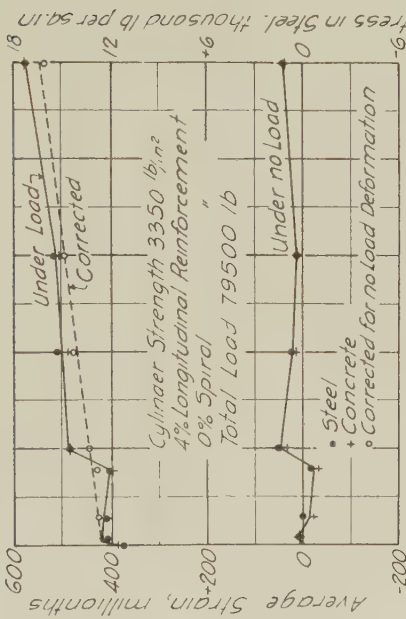


FIG. 19 (LEFT)—DEFORMATION OF COLUMNS 13 AND 14 UNDER SUSTAINED LOAD AND OF COLUMNS 15 AND 16 UNDER NO LOAD IN DRY STORAGE

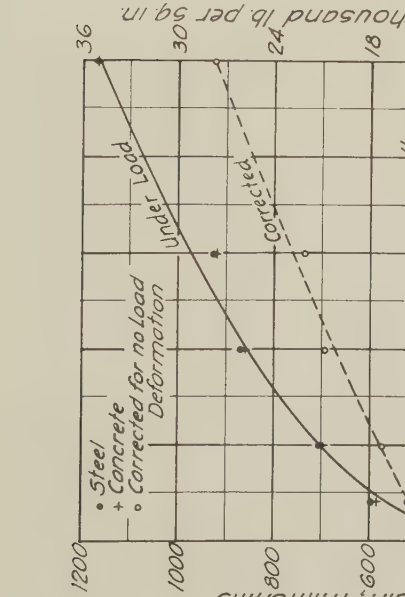


FIG. 20 (ABOVE)—DEFORMATION OF COLUMNS 17 AND 18 UNDER SUSTAINED LOAD AND OF COLUMNS 19 AND 20 UNDER NO LOAD IN WET STORAGE

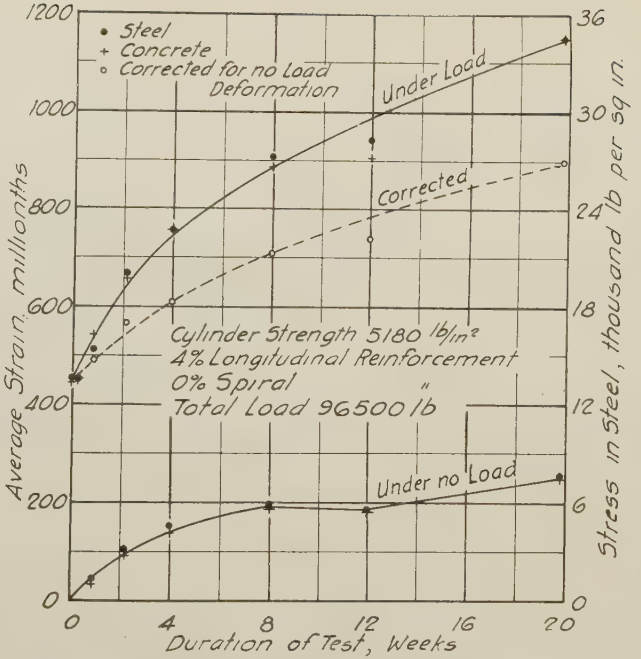


FIG. 21—DEFORMATION OF COLUMNS 21 AND 22 UNDER SUSTAINED LOAD AND OF COLUMNS 23 AND 24 UNDER NO LOAD IN DRY STORAGE

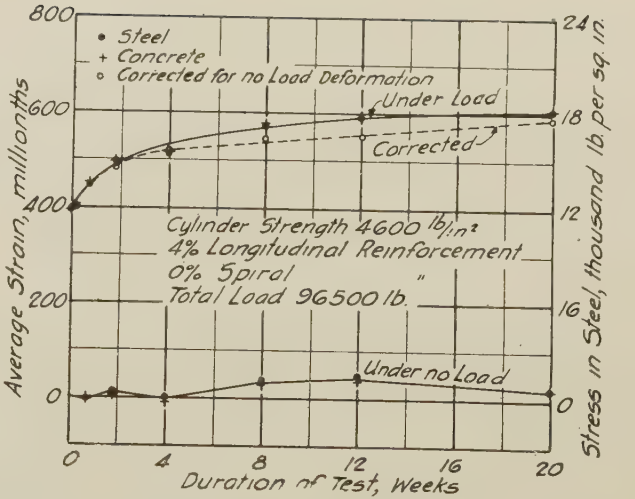


FIG. 22—DEFORMATION OF COLUMNS 25 AND 26 UNDER SUSTAINED LOAD AND OF COLUMNS 27 AND 28 UNDER NO LOAD IN WET STORAGE

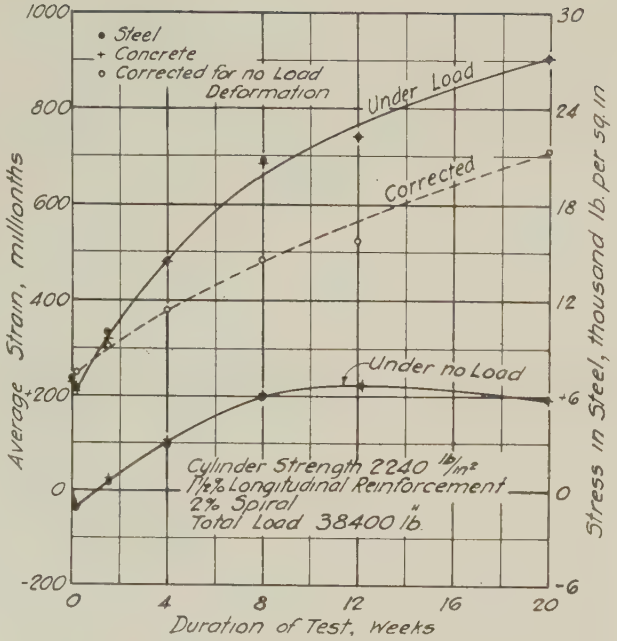


FIG. 23—DEFORMATION OF COLUMNS 29 AND 30 UNDER SUSTAINED LOAD AND OF COLUMNS 31 AND 32 UNDER NO LOAD IN DRY STORAGE

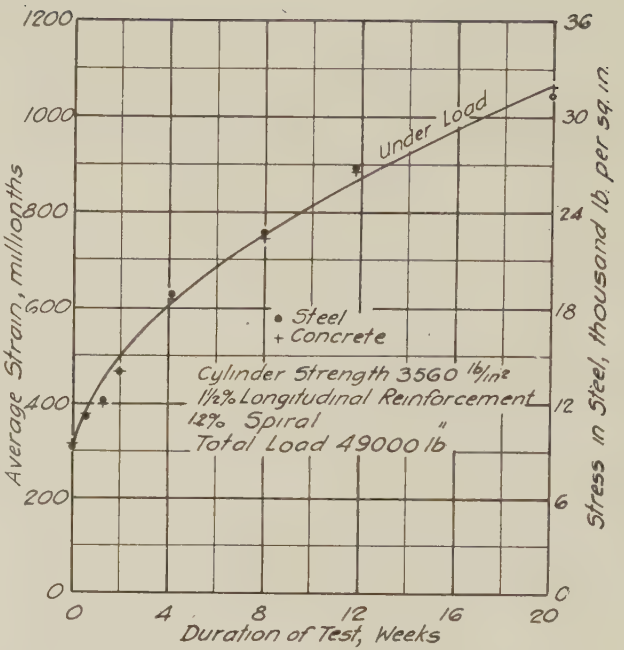


FIG. 24—DEFORMATION OF COLUMNS 33 AND 34 UNDER SUSTAINED LOAD IN DRY STORAGE

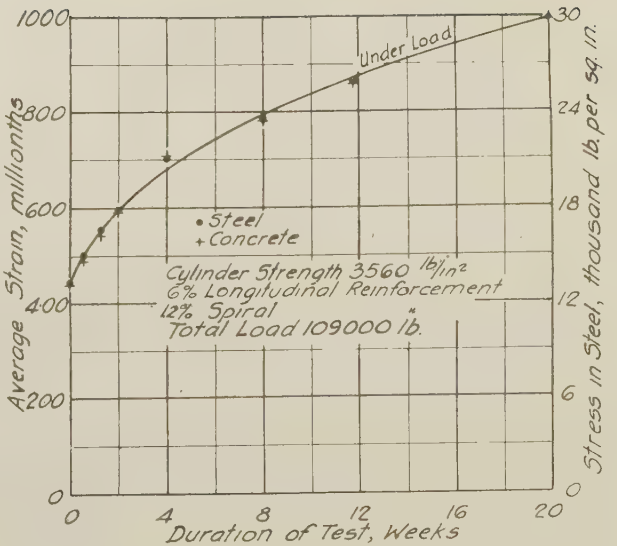


FIG. 25 (ABOVE)—DEFORMATION OF COLUMNS 35 AND 36 UNDER SUSTAINED LOAD IN DRY STORAGE

FIG. 27 (BELOW)—DEFORMATION OF COLUMNS 41 AND 42 UNDER SUSTAINED LOAD AND OF COLUMNS 43 AND 44 UNDER NO LOAD IN DRY STORAGE

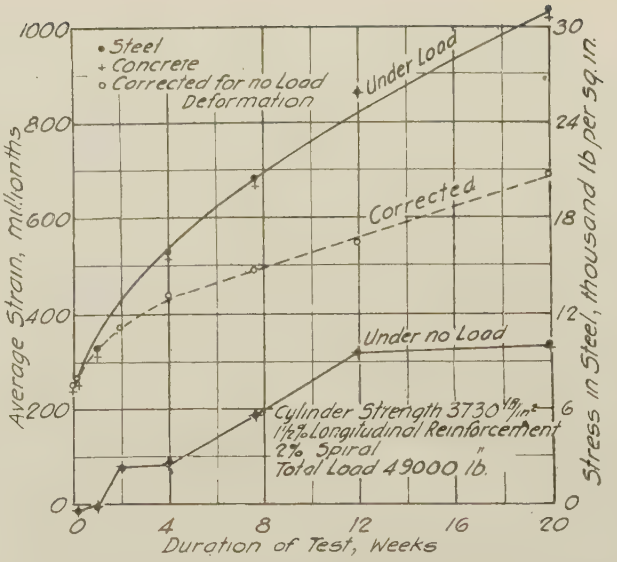
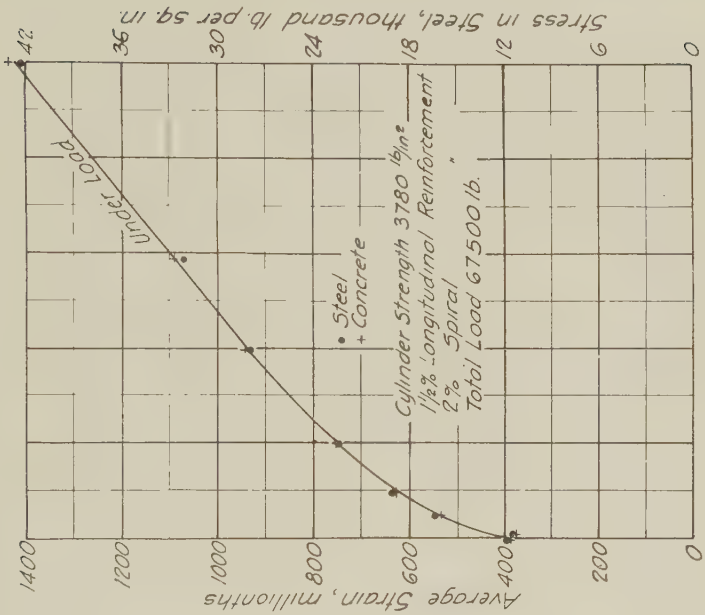
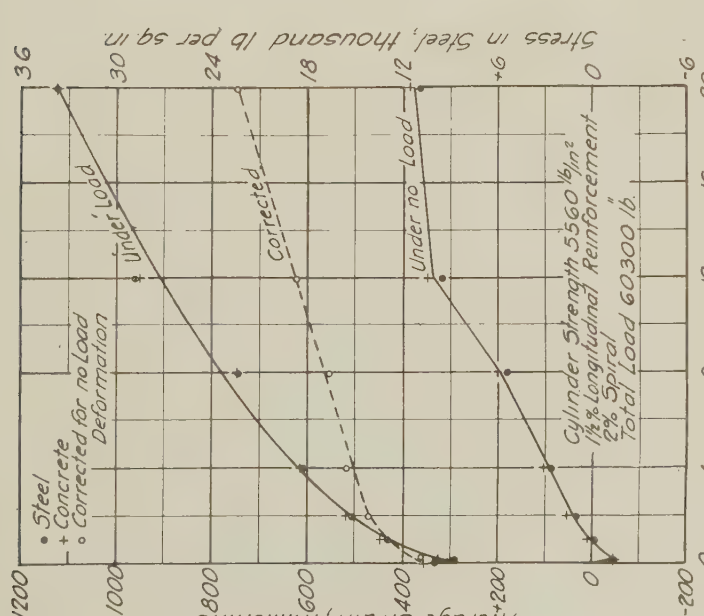


FIG. 26 (ABOVE)—DEFORMATION OF COLUMNS 37 AND 38 UNDER SUSTAINED LOAD AND OF COLUMNS 39 AND 40 UNDER NO LOAD IN DRY STORAGE

FIG. 28 (BELOW)—DEFORMATION OF COLUMNS 45 AND 46 UNDER SUSTAINED LOAD IN DRY STORAGE


Average Strain, millionths
Stress in Steel, thousand lb. per sq. in.
Duration of Test, Weeks
• Steel
+ Concrete
Under Load
Cylinder Strength 3780 lb/in²
1½% Longitudinal Reinforcement
2% Spiral
Total Load 67500 lb.


Average Strain, millionths
Stress in Steel, thousand lb. per sq. in.
Duration of Test, Weeks
• Steel
+ Concrete
○ Corrected for no Load Deformation
Under Load
Corrected
Under no Load
Cylinder Strength 5560 lb/in²
1½% Longitudinal Reinforcement
2% Spiral
Total Load 60300 lb.

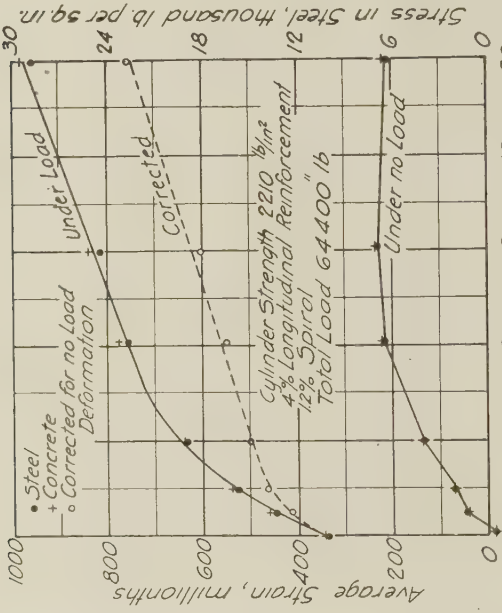


FIG. 30—DEFORMATION OF COLUMNS 49 AND 50 UNDER SUSTAINED LOAD AND OF COLUMNS 51 AND 52 UNDER NO LOAD IN DRY STORAGE

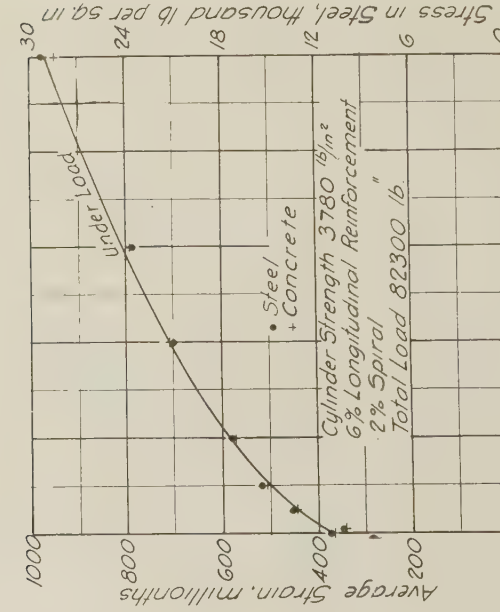


FIG. 29—DEFORMATION OF COLUMNS 47 AND 48 UNDER SUSTAINED LOAD IN DRY STORAGE

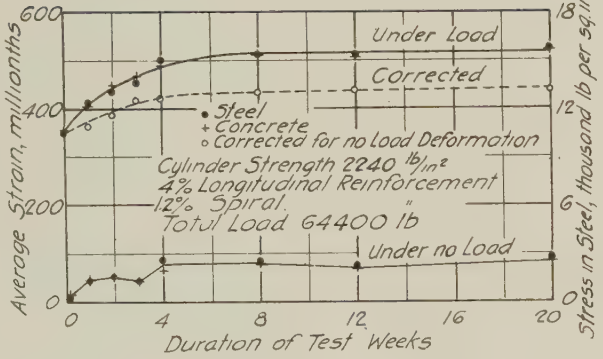


FIG. 31—DEFORMATION OF COLUMNS 53 AND 54 UNDER SUSTAINED LOAD AND OF COLUMNS 55 AND 56 UNDER NO LOAD IN WET STORAGE

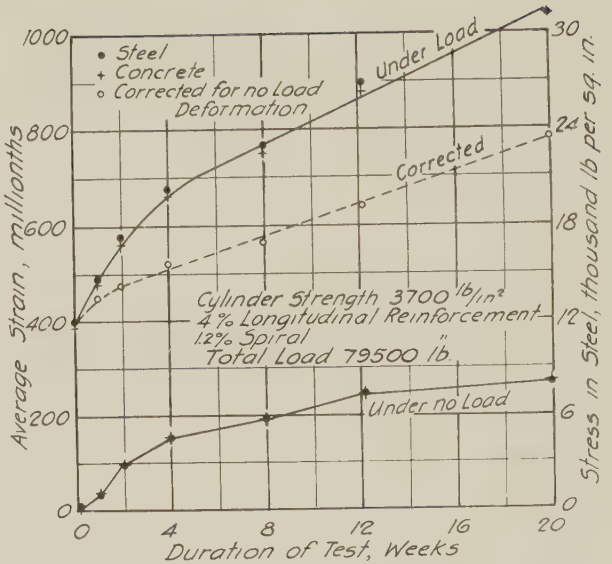


FIG. 32—DEFORMATION OF COLUMNS 57 AND 58 UNDER SUSTAINED LOAD AND OF COLUMNS 59 AND 60 UNDER NO LOAD IN DRY STORAGE

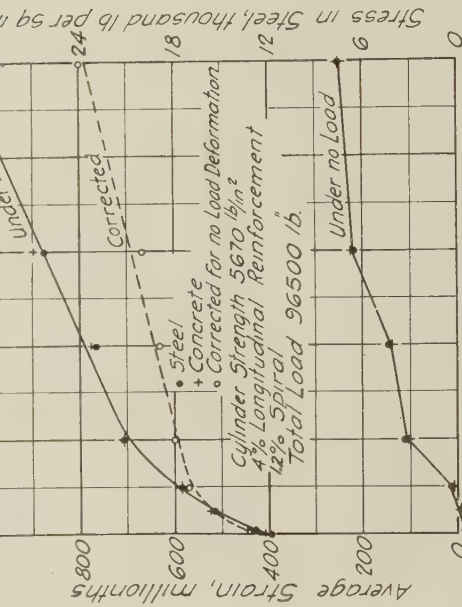


FIG. 34—DEFORMATION OF COLUMNS 65 AND 66 UNDER SUSTAINED LOAD AND OF COLUMNS 67 AND 68 UNDER NO LOAD IN DRY STORAGE

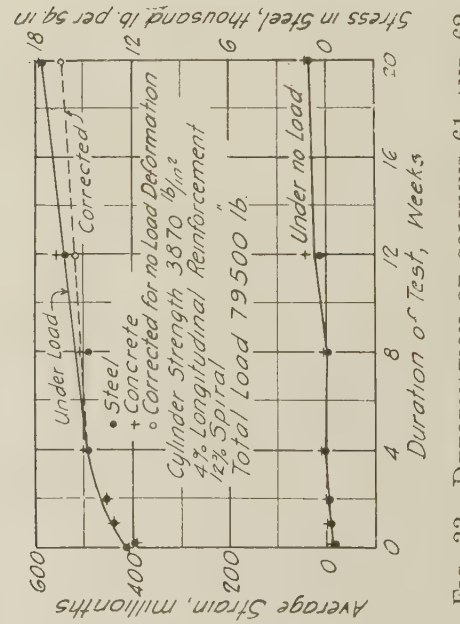


FIG. 33—DEFORMATION OF COLUMNS 61 AND 62 UNDER SUSTAINED LOAD AND OF COLUMNS 63 AND 64 UNDER NO LOAD IN WET STORAGE

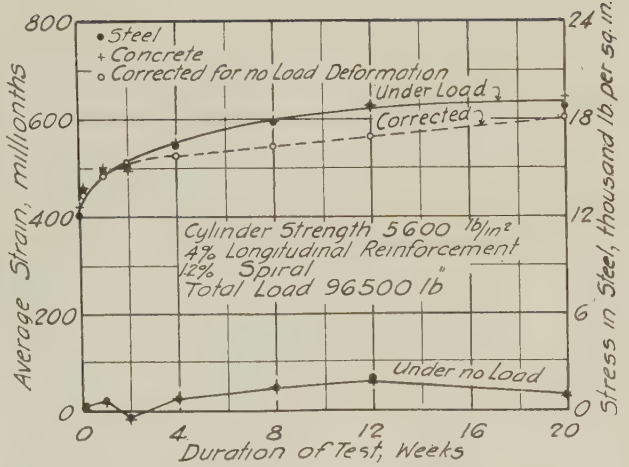


FIG. 35—DEFORMATION OF COLUMNS 69 AND 70 UNDER SUSTAINED LOAD AND OF COLUMNS 71 AND 72 UNDER NO LOAD IN WET STORAGE

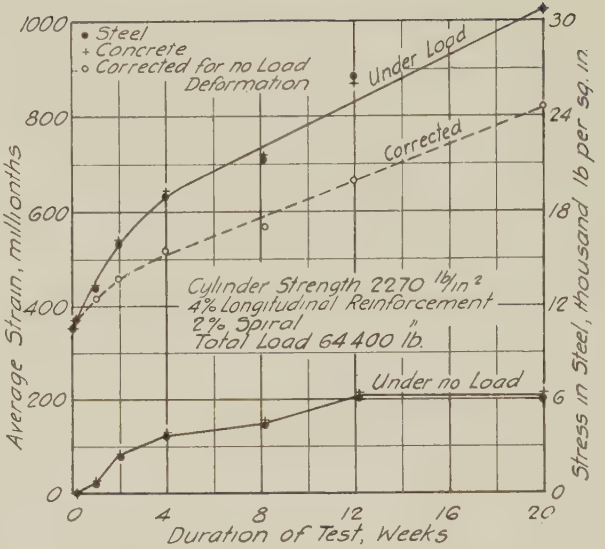


FIG. 36—DEFORMATION OF COLUMNS 73 AND 74 UNDER SUSTAINED LOAD AND OF COLUMNS 75 AND 76 UNDER NO LOAD IN DRY STORAGE

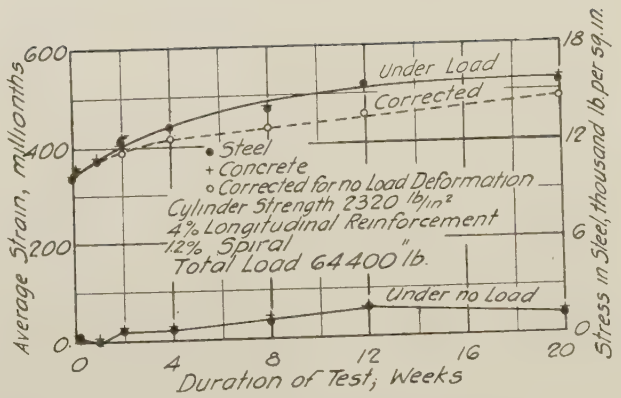


FIG. 37—DEFORMATION OF COLUMNS 77 AND 78 UNDER SUSTAINED LOAD AND OF COLUMNS 79 AND 80 UNDER NO LOAD IN WET STORAGE

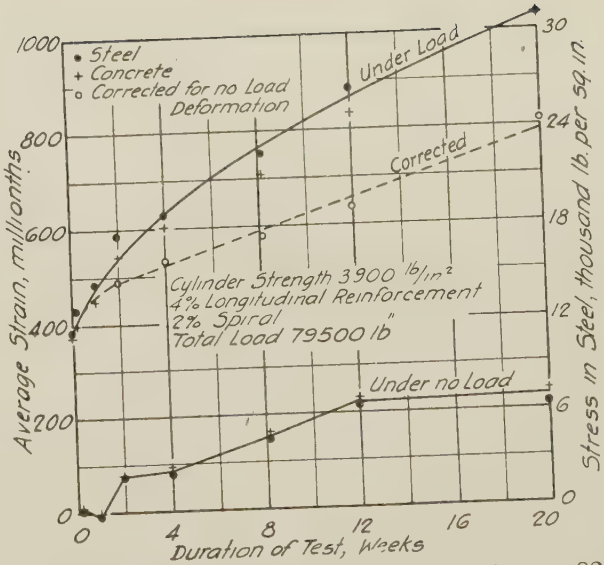


FIG. 38—DEFORMATION OF COLUMNS 81 AND 82 UNDER SUSTAINED LOAD AND OF COLUMNS 83 AND 84 UNDER NO LOAD IN DRY STORAGE

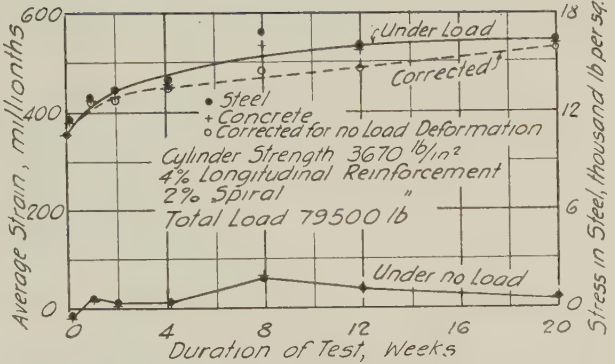


FIG. 39—DEFORMATION OF COLUMNS 85 AND 86 UNDER SUSTAINED LOAD AND OF COLUMNS 87 AND 88 UNDER NO LOAD IN WET STORAGE

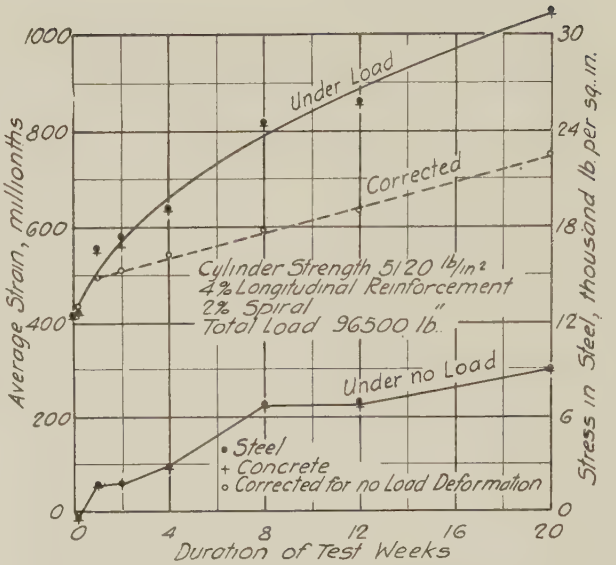


FIG. 40—DEFORMATION OF COLUMNS 89 AND 90 UNDER SUSTAINED LOAD AND OF COLUMNS 91 AND 92 UNDER NO LOAD IN DRY STORAGE

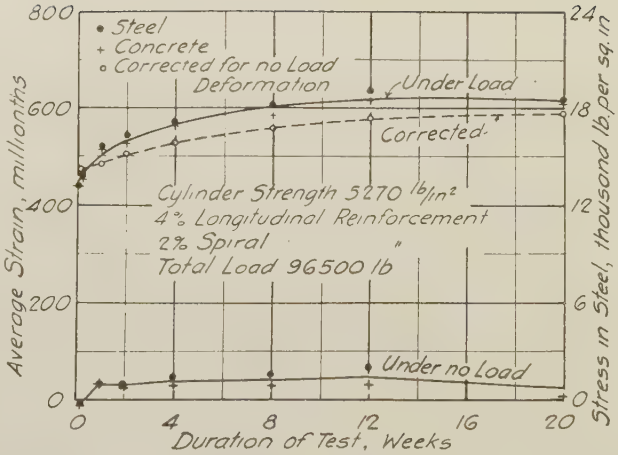


FIG. 41—DEFORMATION OF COLUMNS 93 AND 94 UNDER SUSTAINED LOAD AND OF COLUMNS 95 AND 96 UNDER NO LOAD IN WET STORAGE

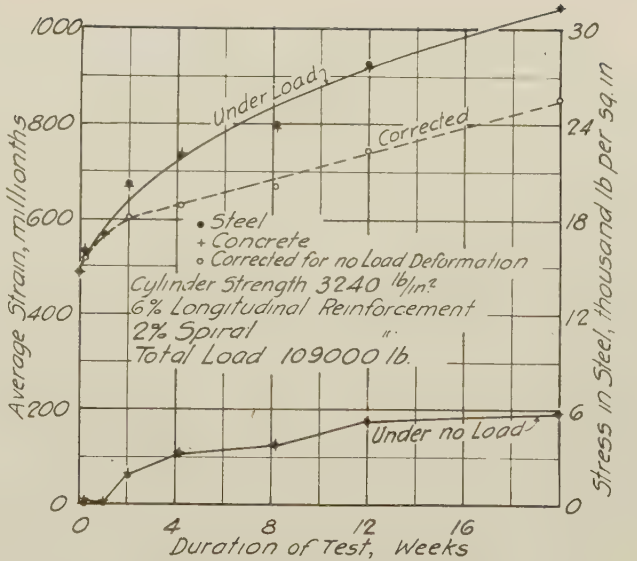


FIG. 42—DEFORMATION OF COLUMNS 97 AND 98 UNDER SUSTAINED LOAD AND OF COLUMNS 99 AND 100 UNDER NO LOAD IN DRY STORAGE

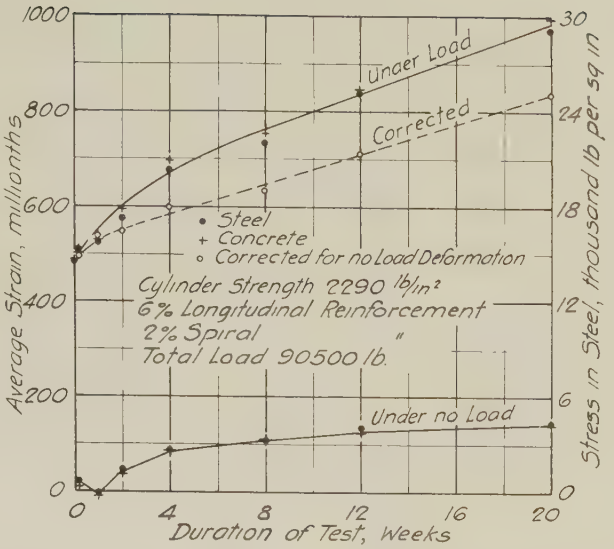


FIG. 43—DEFORMATION OF COLUMNS 101 AND 102 UNDER SUSTAINED LOAD AND OF COLUMNS 103 AND 104 UNDER NO LOAD IN DRY STORAGE

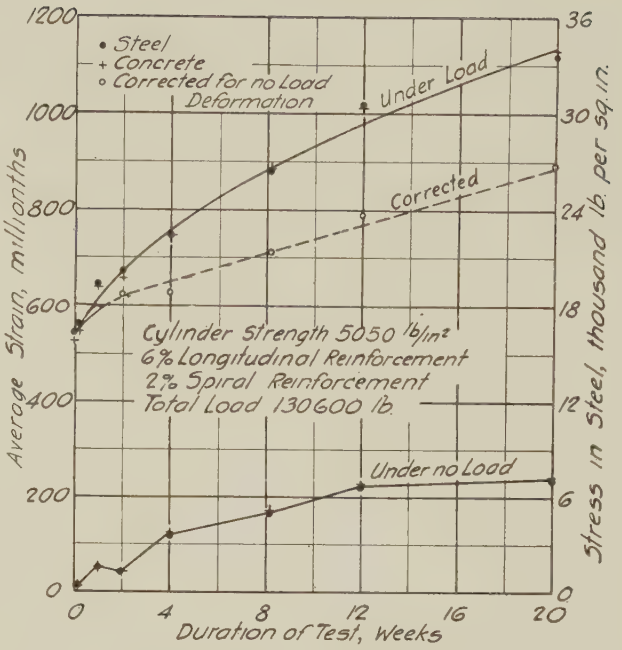


FIG. 44—DEFORMATION OF COLUMNS 105 AND 106 UNDER SUSTAINED LOAD AND OF COLUMNS 107 AND 108 UNDER NO LOAD IN DRY STORAGE

Flow of Concrete under the Action of Sustained Loads

BY RAYMOND E. DAVIS* AND HARMER E. DAVIS†

This paper is here published to encourage discussion at the 27th annual convention at which it will be presented formally in abstract. It represents a part of the work assigned to Committee 109—Summary of Plastic Flow. Professor Davis is Author-Chairman and Harmer E. Davis is a member of the committee. It is solely their joint effort in which the committee as a whole has not participated.—EDITOR

SYNOPSIS

THIS PAPER presents the results of tests on plain and reinforced concrete to determine the effect of loads sustained over a considerable period of time upon the deformations and upon the distribution of stresses in concrete and steel.

Part 1 of the paper describes laboratory tests at the University of California in the interval 1925-1930 on more than 200 cylinder specimens, all maintained under controlled conditions of storage as regards temperature and humidity, to determine the influence of a constant sustained compressive stress upon axial deformations. The deformation due to stress which takes place immediately upon the application of the load is here called the "instantaneous deformation," and that which takes place subsequent to the application of load is here designated as the "flow," a quantity separate from changes in dimensions due to such causes as variations in temperature and moisture conditions. Likewise the ratio of unit stress to unit instantaneous deformation is termed the "modulus of elasticity," and the ratio of the sustained unit stress to the total deformation due to load (instantaneous plus flow), is called the "sustained modulus of resistance."

The tests were made to determine the manner in which flow is influenced by such elements as (1) length of period of sustained load, (2) magnitude of sustained load, (3) age at the time of loading, (4) character of mineral aggregate, (5)

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richness of mix, (6) gradation of aggregate, (7) water-cement ratio, (8) reinforcement, (9) alternately applying and releasing load, and (10) moisture conditions of storage, including variations in humidity of surrounding air.

From these tests, it has been determined among other things that the flow of plain concrete under a constantly sustained compressive stress (1) continues to increase, at first rapidly and then more slowly, for a long time, certainly for several years, (2) is greater for concrete in an atmosphere of low humidity than for an identical concrete in an atmosphere of high humidity or when fully saturated with water, (3) is materially affected by the character of the aggregate, being substantially larger for sandstone concrete than for limestone concrete, (4) is less for rich mixes than for lean ones, (5) is less for concrete having a high modulus of elasticity than for one having a low modulus of elasticity, (6) is greater for a high water-cement ratio than for a low one, and (7) is greater when load is applied at an early age than when it is applied at a later age.

Longitudinal reinforcement in specimens stored in air appreciably reduces both the flow and shrinkage, but under the action of a load long sustained, the compressive stress in the longitudinal steel continues to increase until it becomes several times the stress which existed immediately upon application of the load, even reaching the yield point of the steel.

When after a time the load is released, there is an elastic recovery which is immediate, and a plastic recovery which increases with time, but the plastic recovery is less than the plastic flow which took place under the sustained loading. When concrete has been so loaded and unloaded, the instantaneous modulus of elasticity becomes somewhat greater than that of a concrete alike in all other respects save that it has not been previously stressed.

In time, even at normal working stresses, the flow of concrete may be several times as great as the initial instantaneous deformation, or, putting it in another way, the instantaneous modulus of elasticity is several times the sustained modulus of resistance.

When the load is alternately applied and released for a given time, the magnitude of the instantaneous deformation and the rate of plastic flow and plastic recovery tend to become less as the number of stress cycles is increased.

Part 2 of the paper summarizes the published results of the more important tests reported by other investigators during the last 25 years, in which the phenomenon of "time yield," "plasticity," "time deformation," "plastic flow" or "flow" has been observed and discussed. In general, while the conditions surrounding these early tests are not sufficiently well known nor the tests sufficiently extensive to make the quantitative data of direct value, the results consistently point to a marked time effect, both upon laboratory specimens and upon service structures.

Tests which are here summarized include those reported by Hatt in 1907 on beams; by McMillan in 1915, 1916 and 1921, on beams, slabs, columns; by Fuller and More in 1915, on cylinders and flat slabs; by Goldbeck and Smith in 1914-1917, on cylinders, beams and slabs; by Lord in 1914-16, on flat slabs; and by Faber, 1916-27, on beams.

INTRODUCTION

FOR MORE than a quarter of a century the property possessed by all concrete, to a greater or less degree, of yielding slowly under the action of sustained loads has been known to exist, first through the gradually increasing deflection of beams and slabs and later through measurements of the strains in both concrete and steel of columns, beams and slabs. This property, which is entirely separate from shrinkage due to gradual release of uncombined water when concrete is in air, renders concrete, even within working stresses, not a perfectly elastic material, but a material which when subjected to sustained load continues slowly to yield or deform for a long time. That is, not only does the stress-strain relation of a given concrete vary with the magnitude of the stress, but also with the length of time over which the stress is sustained.

This property of yielding with the passage of time has been designated as "time yield," "time deformation," "plastic deformation," and "plastic flow." In this report, except in quoting the words of other investigators, the term "flow" will be employed, though it is problematical whether there actually occurs in concrete a movement of the particles over one another in the manner commonly associated with the flow of any material.

In many structures where dead load forms a large part of the total load this slow yielding of the concrete with time would seem to bring about a stress distribution quite different from that which would exist under the action of an equivalent live load. In buildings and other structures exposed to drying conditions there also enters further to complicate the problem the added factor of shrinkage of the concrete, which likewise is gradually accumulative over a long period. Take the simple case of a reinforced column, for example; due to dead load, the stress in the concrete may gradually diminish and the stress in the longitudinal steel gradually increase as flow and shrinkage take place, until the yield point of the steel is reached, after which the distribution of stress between steel and concrete may perhaps remain substantially constant. Or consider a simply supported reinforced beam, in which the flexural stress is insufficient to produce cracks below the neutral axis and which is free to contract longitudinally;

due to shrinkage alone the steel is gradually placed in compression, with the top fibers of the beam in compression and the bottom fibers in tension, and due to dead load deformations there may be gradually produced a lowering of the neutral axis tending to increase the tension in the steel and to decrease the compression in the extreme fiber of the concrete. Thus the effect of flow may be to decrease the stress in the concrete and the effect of shrinkage may be to decrease the tensile stress in the steel. In restrained beams, continuous frames, and arches it appears likely that flow acts in some degree to relieve the stresses at sections of large moment and to redistribute the moment in a manner more nearly to equalize negative and positive moments. For example, it might be expected that for a beam fixed at the ends and carrying a uniform dead load, the points of contra flexure would gradually shift position and the elastic curve gradually change its shape until, in the process of time, the maximum positive and negative moments were substantially the same. Or for an arch where there was a gradual rib shortening, due to whatever cause, it appears probable that the moments at crown and spring lines may be substantially less than would be the case were the length of the arch ring abruptly changed by the same amount. And the dead-load moment diagram for a continuous beam with concentrated loads long sustained, instead of being a series of connected straight lines, may be a curved line with summits and depressions rounded.

Many other examples might be cited, but those mentioned are sufficient to indicate the desirability of determining the facts concerning flow and of considering the influence of flow upon structural design. The recognized importance of flow as a factor in stress analysis has prompted the preparation of this paper. Here attempt is made to present quantitative data concerning the phenomenon of flow as determined by tests, leaving to others the task of discussing the significance of flow as related to stresses in structures.

The paper is divided into two parts:

Part 1 describes in considerable detail tests made in the interval 1925-30 at the University of California under the direction of the

authors upon more than 200 compression cylinders under closely controlled conditions. Here the influence of various factors upon flow are discussed.

Part 2 is a summary of some of the more important papers and discussions published in the last 25 years in which the effects of flow, as determined by tests, have been described. These include laboratory tests on cylinders, beams and slabs, and field observations upon the several structural elements. Following the abstracts is a bibliography of articles and papers dealing with flow and shrinkage.

In this paper the term "flow" will be taken to mean that axial deformaton due to stress which takes place subsequent to the application of a sustained load; that is, the flow at the end of any interval of time is the deformation due to the sustained load but not to the instantaneous application of the load. It is the total deformation due to stress, minus the deformation which takes place immediately upon the application of the load. It does not include changes in length due to causes other than stress, such as shrinkage and variations in temperature.

Since concrete under the action of sustained stress is so imperfectly elastic, it seems particularly inappropriate to use the term "modulus of elasticity" as being the ratio of unit stress to unit deformation when flow occurring with the passage of time is included. Consequently in this report the term "modulus of elasticity" will be employed only to designate the ratio of unit stress to unit strain at the time of application of load, and the term "modulus of resistance" will be employed to designate the ratio of unit stress to total unit deformation, this total to include the "instantaneous deformation" occurring immediately upon the application of load, plus the deformation, due to sustained stress, occurring subsequent to the application of load.

Just as concrete instantaneously deforms upon application and continues to deform for a long period if the load is sustained, so upon release of load is there an immediate reduction in deformation, here called the "instantaneous recovery," followed by a further gradual reduction in deformation which extends over a considerable period, here designated as the "plastic recovery."

PART 1—UNIVERSITY OF CALIFORNIA TESTS

Remarks

The tests here reported were conducted in the Materials Testing Laboratory of the University of California to determine the behavior of concrete under long continued compressive stress. From a modest beginning in 1925, the program has been periodically enlarged until in 1930 the number of specimens had reached a total of 217, of which 131 were subjected to sustained stress and 86 were maintained in an unstressed condition, being under observations to determine dimensional changes (shrinkage or expansion) due to causes other than stress. As this paper is written, a considerable number of the specimens have been under observation for more than four years.

All tests have been carried out under closely controlled conditions, the specimens being constantly maintained at 70 deg. F. and the atmospheric humidity being constantly maintained at a desired value. In this manner the uncertainties arising from variable temperature and humidity have been eliminated or the effects of known variations in the humidity have been observed.

For every specimen which has been subjected to sustained stress, there has been a similar specimen maintained under identical conditions except that it has been subjected to no load. These have been designated as "controls," for by means of them it has been possible to determine the *flow* of each loaded specimen by deducting from the total observed deformation, the observed shrinkage or expansion of its *control*.

The specimens have been cylinders varying from 4- to 10-in. diameter and from 12- to 24-in. length. Among the specimens a constant sustained compressive stress as low as 200 lb. per sq. in., and as high as 1200 lb. per sq. in. has been maintained, and the ages at the time of loading have varied from 3 days to 7 months.

Acknowledgments

The tests here described have been made possible through the generous financial support rendered by the Research Board of the University of California, by Engineering Foundation, and by the American Society of Civil Engineers, through its Committee on Arches.

In the years since the tests were started more than a dozen men at one time or another have contributed their services to the work, and to this group thanks are due to the painstaking manner in which the tests have been executed. In particular it is desired to acknowledge the valuable assistance rendered by G. E. Troxell, Associate Professor of Civil Engineering, who supervised many of the earlier tests; J. L. Banville, Assistant in the Materials Testing Laboratory who has been immediately in charge of much of the work since its beginning and who in a large measure is responsible for the design of special instruments and apparatus; and J. S. Hamilton, Assistant in the Materials Testing Laboratory, who has reviewed and assembled many of the data and who has supervised many of the later tests.

Scope of the Tests

The test program has been designed to show the effect upon the time-flow relation of the following:

1. Richness of mix,
2. Gradation of the aggregate,
3. Mineral composition of the aggregate,
4. Age at time of applying load,
5. Condition of storage as regards moisture,
6. Magnitude of stress,
7. Reinforcement,
8. Alternately applying and releasing loads.

Also, tests have been made to determine the effect of flow upon the strength and elasticity.

The tests are divided into seven series, as follows:

Series 1—Inaugurated in 1925 to determine the effect of richness of mix and gradation of the aggregate upon flow. Completed in 1926.

Series 2—Inaugurated in 1926 to determine the effect of magnitude of unit stress, age at time of loading and moisture conditions upon flow. Tests still in progress.

Series 3—Inaugurated in 1928 to determine the effect of humidity upon flow. Still in progress.

Series 4—Inaugurated in 1928 to determine the effect of character of the mineral aggregate upon flow. Still in progress.

Series 5—Inaugurated in 1929 to determine the effect of reinforcement upon flow and flow upon stress in reinforcement. Still in progress.

Series 6—Inaugurated in 1929 to study the effect of alternately applying and releasing a constant sustained load upon flow. Still in progress.

Series 7—Inaugurated in 1929 to study the effect of richness of mix and moisture conditions upon flow and to determine the effect of flow upon the modulus of elasticity and compressive strength. Completed December 1930.

Further details concerning the several series follow:

Series 1

The purpose of these tests was to determine the effect upon flow of (1) richness of mix, (2) gradation of aggregate, and (3) water-cement ratio.

The specimens were 6 by 24-in. cylinders, 16 in number, 12 of which were subjected to a continuous compressive stress of 640 lb. per sq. in. and 4 of which were maintained in an unstressed condition. All specimens were cured for 2 months in water and 5 months in air prior to the application of load. After the beginning of the loading tests, all specimens, including those of the unstressed group, were stored in an atmosphere of 70 deg. F. and 78 per cent relative humidity.

Four concrete mixes were investigated, differing from one another either in regard to richness or to gradation of aggregate, all as indicated in Table 1.

TABLE 1—PROPORTIONS—SERIES 1

Mix	Cement-Aggregate Ratio		Fineness Modulus of Aggregate	Water-Cement Ratio by Volume
	By Volume	By Weight		
1	1:3.4	1:4	6.43	0.81
2	1:3.4	1:4	3.62	1.06
3	1:5.9	1:7	6.43	1.01
4	1:5.9	1:7	3.62	1.42

For mixes of low fineness modulus (mixes 2 and 4) the aggregate was composed of Niles top gravel, a Coast Range sand all of which passed a No. 4 sieve. For mixes of high fineness modulus the fine aggregate was Niles top gravel and the coarse aggregate was a crushed metamorphic sandstone between the sizes ½ and 1½ in. from the local quarries of the Oakland Paving Co. From each mix there were manufactured four specimens, three of which were loaded and one of which maintained without stress.

Series 2

The purpose of these tests was to determine the effect of (1) magnitude of sustained stress, (2) age at time of loading, and (3) air *versus* water storage.

The specimens were 4 by 14-in. cylinders, 58 in number, 40 of which were loaded and 18 of which were unstressed.

The concrete was the same for all specimens, being composed of 1 part cement to 5.05 parts, by weight, of an aggregate composed entirely of crushed granite for which the fineness modulus was 5.03. The proportioning and materials were the same as those employed in the concrete used in the construction of the Stevenson Creek Test Dam of the Arch Dam Investigation of Engineering

Foundation. The water-cement ratio was 1.03 by volume and the average slump was 3.2 in. The mix data are given in Table 2.

TABLE 2—PROPORTIONS—SERIES 2

	Cement	Fine Aggregate	Medium Aggregate	Coarse Aggregate
Proportions by Weight	1	3.01	1.15	0.89
Sieve sizes		Below ⅜ in.	⅜ to ¾ in.	¾ to 1½ in.
Fineness Modulus		3.27	6.72	8.78

In Table 3 are given the storage and stress conditions investigated. Examining the table it will be found that the age at time of loading varied from 2 days to 3 months and that the intensity of sustained stress varied from 200 to 1200 lb. per sq. in. In general, observations were made upon 3 control specimens for each age of loading and storage conditions, and observations were made upon 3 loaded specimens for each age of loading. storage condition and intensity of sustained stress. For those specimens stored in water, the

TABLE 3—STORAGE AND STRESS CONDITIONS OF SPECIMENS—SERIES 2

Curing	Storage After Loading	Age at Loading	Stress, Lb. Per Sq. In.
Water	Water	2 days	200
Water	Water	7 days	300 600
Damp Sand	Water	28 days	300 600 900
Damp Sand	Water	3 months	600 900 1200
Damp Sand	Air at 70 per cent relative humidity	28 days	300 600 900
Damp Sand	Air at 70 per cent relative humidity	3 months	600 900 1200

temperature of the bath was 70 deg. F., and for those stored in air the temperature of the surrounding atmosphere was 70 deg. F. and the relative humidity was 70 per cent.

Series 3

The purpose of these tests was to determine the effect upon flow of variations in the conditions of storage with regard to moisture. The conditions of storage which were included in the investigation were (1) air at 50 per cent relative humidity, (2) air at 70 per cent relative humidity, (3) fog, and (4) under water.

The specimens were 4 by 14-in. cylinders, 20 in number, 12 of which were loaded and 8 of which were used as unloaded controls.

The concrete was the same for all specimens, being composed of 1 part of cement to 5.67 parts by weight of Niles gravel. The mix data are given in the following table. The fineness modulus of the mixed aggregate was 5.61 and the water-cement ratio was 0.89.

TABLE 4—PROPORTIONS—SERIES 3

	Cement	Fine Aggregate	Medium Aggregate	Coarse Aggregate
Proportions by Weight	1.00	2.00	1.67	2.00
Fineness Modulus		3.48	5.99	7.40

The specimens were fog-cured for 28 days when those carrying a sustained stress were all loaded to 800 lb. per sq. in. For each of the four conditions of storage there were three loaded and two control specimens.

Series 4

The purpose of these tests was to determine the effect of the character of mineral aggregate upon flow. The aggregates which were included in the investigations included (1) Niles gravel, containing quartz, sandstones, granites, etc., (2) metamorphic sandstone from Oakland, Calif., (3) quartz, (4) basalt from Napa, Calif., (5) limestone from Santa Cruz, Calif., and (6) granite.

The specimens were 4 by 14-in. cylinders, 30 in number, of which 18 were loaded and 12 were used as controls.

The concrete was of the same richness of mix for all specimens, namely, 1 part of cement to 5.67 parts by weight of mixed aggregate. The water-cement ratio was uniformly 0.89 by volume.

For the gravel concrete, the mix and gradation were as indicated under Series 3. Other aggregates were crushed, screened in a variety of sizes, and then these sizes recombined to give a sieve analysis curve practically the same as that of the gravel. That is, the gradation of one mixed aggregate was the same as for all others. The fineness modulus of each mixed crushed aggregate was 5.75.

The consistency of the fresh concrete for the several aggregates, as determined by slump and flow tests was as shown in Table 5.

The specimens were fog-cured until loaded at the age of 28 days and were thereafter stored in air at 50 per cent relative humidity. For each aggregate, three specimens were loaded to a stress of 80 lb. per sq. in. and two specimens were employed as unstressed controls.

Series 5

The purpose of these tests was to determine the effect of steel reinforcement upon flow. The specimens are 10 by 20-in. cylinders, 4 in number, 2 of which are

TABLE 5—CONSISTENCY—SERIES 4

Aggregate	Slump., In.	Flow
Gravel	4	141
Sandstone	2¼	138
Quartz	4¼	156
Basalt	3	137
Limestone	2	137
Granite	3½	142

plain and 2 of which are reinforced with longitudinal bars and spiral hooping. The reinforcement consists of six ½-in. square deformed bars, and an 8-in. diameter spiral of No. 6 A. S. W. G. of 1¼ in. pitch. The steel is of structural grade and the ends of the longitudinal bars are ground flush with the ends of the cylinders. Considering the core area, the steel ratio for longtiudinal bars is 0.0298 and for spiral is 0.0133. Considering the entire cross-section of the cylinder, the steel ratio for the longitudinal bars is 0.0190.

The concrete is composed of 1 part of cement, to 2.64 parts Niles sand, to 2.32 parts crushed sandstone, all by volume. The fineness modulus of the combined aggregate was 5.22 and the water-cement ratio was 0.86. The slump was 2 in.

The specimens were cured in fog for 50 days and thereafter were stored in air at 50 per cent relative humidity. At the age of 60 days, one plain and one reinforced specimen were loaded. The stress in the plain concrete was 800 lb. per sq. in. and the stress in the reinforced concrete at the time of loading was calculated to have been 775 lb. per sq. in. on the full concrete area.

Series 6

The purpose of these tests was to determine the behavior of concrete under alternations of a maximum and a minimum sustained stress. Observations were made to determine the instantaneous and time yields each time the load was increased from minimum to maximum and the instantaneous and time recoveries each time the load was decreased from the maximum to the minimum.

The specimens are 4 by 14-in. cylinders, 30 in number, 18 of which are subjected to load alternations and 12 of which are unstressed control specimens.

The concrete was the same for all specimens. The mix by weight was composed of 1 part of cement to 5.68 parts of mixed aggregate. The mix data are given in Table 6. The fineness modulus of the combined aggregate was 4.86 and the water-cement ratio was 0.85. The slump was 4 in.

TABLE 6—PROPORTIONS—SERIES 6

	Cement	Monterey Sand	Niles Sand	Niles Gravel
Proportions by Weight.....	1	1.14	1.14	3.40
Fineness Modulus.....		1.74	2.90	6.72

The specimens were fog-cured for 28 days, when load alternations were begun. Thereafter half the specimens were stored in air at 50 per cent relative humidity and the remainder were stored under water.

For all specimens the maximum sustained stress was 800 lb. per sq. in. and the minimum sustained stress was 40 lb. per sq. in. A stress alternation or cycle consisted of a given period at the maximum stress followed by a like period at the minimum stress.

In Table 7 are given the lengths of stress cycle for the several groups of specimens.

TABLE 7—LENGTH OF STRESS CYCLES—SERIES 6

Storage Conditions	Length, Stress Cycle, Days	Group Designation	No. of Specimens	
			Loaded	Control
Air 50% humidity	2	C	3	3
Air 50% humidity	10 to 14	B	3	3
Air 50% humidity	60	A	3	
Water	2	C	3	3
Water	10 to 14	B	3	3
Water	60	A	3	

Series 7

The purpose of these tests was to study the effect of richness of mix and storage conditions upon flow and the effect of flow upon the compressive strength and the modulus of elasticity, that is, it was desired to learn how these properties of a concrete which had been subjected to a sustained stress for a considerable period of time would compare with those of an identical concrete not previously stressed.

The specimens were 6 by 12-in. cylinders, 64 in number, half of which were placed under continuous load at 28 days and the remainder of which were not stressed prior to the strength test.

Two concrete mixes were employed, a rich mix composed of 1 part cement to 3.6 parts by volume of combined aggregate, and a lean mix composed of 1 part cement to 6.3 parts of combined aggregate. For both mixes a constant slump of 3 to 4 in. was used, giving a water-cement ratio which varied slightly for each mix. For the rich mix the fineness modulus of the combined aggregate was 5.70 and the water-cement ratio was close to 0.63 by volume. For the lean mix the fineness modulus of the combined aggregate was 5.55 and the water-cement ratio varied from 0.95 to 1.02 by volume. Table 8 gives the proportions of the separate aggregates.

For the first 7 days all specimens were fog-cured. Thereafter half the specimens were stored under water and the remainder were stored in air at 50 per cent relative humidity. At 28 days, half of those stored in both air and water were loaded to 800 lb. per sq. in. The remainder, as noted above, were left unstressed.

TABLE 8—PROPORTIONS—SERIES 7

Cement-Aggregate, Ratio by Volume	Parts by Weight				
	Cement	Monterey Sand	Top Gravel	Medium Gravel	Coarse Gravel
1:3.6	1	0.66	0.66	1.34	1.34
1:6.3	1	1.26	1.26	2.24	2.24
Fineness Modulus		2.15	2.92	6.88	7.63

At the age of 5 months (a period of 4 months of sustained stress for those specimens which were loaded) half the number of specimens in each condition of storage and of each mix were tested in compression. At the age of 12 months, when the period of sustained stress was 11 months, the remaining specimens of each group were tested. At the ends of these periods the load was removed from the specimens which were subjected to sustained stress, and observations of instantaneous recovery were made. In the 5-months tests, as quickly as possible after unloading, the specimens were tested to failure, in compression, measurement of deformation being made with a compressometer as the loading progressed. In the 12-months tests, after unloading, the plastic recovery was made for four or five days before the elastic compression tests were made.

Testing Apparatus and Methods

Except for the elastic compression tests of Series 7, where an attached dial compressometer was employed, all changes in length have been measured with portable strain gages, those of Series 1 with a 20-in. gage of the multiplying-lever type, and those of all later series with 10-in. gage of the fulcrum-plate type. The latter type has been found particularly satisfactory.

For each specimen observations of changes in length have been made upon three longitudinal gage lines at the third points of the circumference. The reference marks are No. 50 drill holes into which are thrust the points of the strain gage. For Series 1, ½-in. diameter carbon steel plugs were grouted in the proper position into holes drilled in the specimens, the gage distance was punch marked with a templet, and the gage holes were drilled with a hand drill. For all later series ½-in. square brass plugs each threaded to receive a screw, were fastened to the interior of the molds in true position by means of screws passing through the molds. When the concrete had hardened sufficiently the screws were withdrawn and the molds were removed from the cylinders. Later in each brass plug was screwed a rustless steel insert with center hole forming the gage mark. This type of gage marker, consisting of the brass plug with hard-tempered, rustless steel insert, has proven so eminently satisfactory, not only because of its low cost for making and placing, but because of the precision with which the gage holes can be placed and aligned, and because of resistance to wear and corrosion, that attention is invited to its features.

For the specimens of Series 5, in addition to the three longitudinal gage lines with points in the circumferential surface, there are employed two additional

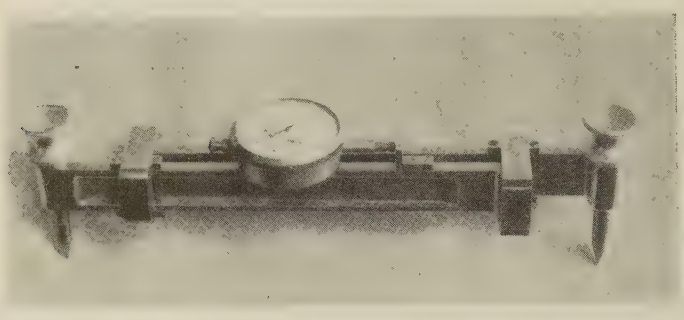


FIG. 1—FULCRUM PLATE STRAIN GAGE

sets of corresponding gage lines, one set of three with plugs set in the concrete at a depth of one inch from the surface, and the other set of three with gage holes in the longitudinal reinforcing bars at 1 in. depth.

In each loaded specimen the load has been maintained constant by means of one or more car springs held in compression by a system of rods and plates between which the specimen is clamped. The desired load is applied by mounting this apparatus in a testing machine with a test specimen in position, the load being indicated by the machine. In order to eliminate the disturbing influence of variable temperature, a small portable machine has been built specially for applying loads to these specimens, and this machine is placed in one of the constant temperature rooms where the specimens are stored. When a specimen is loaded, the nuts on the rods of the load-sustaining apparatus are brought to a bearing against the end plates. The load on the testing machine is then released and the compression in the car spring and in the specimen is maintained by tension in the rods. This method of maintaining stress has proved very satisfactory, since the specimens may be moved about and placed in any position after the load has once been applied, without disturbing stress conditions.

For each group of loaded specimens there is a corresponding group of unstressed or control specimens which is stored in the same conditions as the loaded specimens. Observations made upon these control specimens (made simultaneously with observations on loaded specimens) give changes in length due to causes other than stress, and this makes it possible to determine the flow which has occurred in the loaded specimens as a quantity separate from volume changes due to variations in moisture or temperature.

All specimens have been stored continuously at 70 deg. F. in specially insulated rooms heated by electricity and equipped with sensitive thermostats. Furthermore those stored in air have been placed in rooms where the humidity is controlled as well. Under normal conditions variations from the established standard do not exceed 1 deg. F. in temperature and 1 per cent in relative humidity.

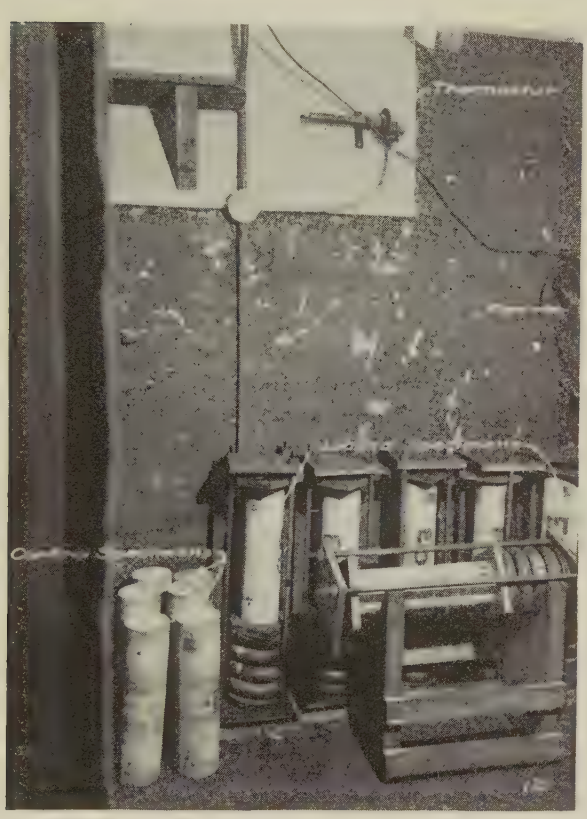


FIG. 2—SPECIMENS IN DRY AIR STORAGE, SERIES 2

Results of Tests

The data of the tests are presented in graphs exhibited as Figs. 4 to 26 inclusive, many of which show the time-flow relation. Each plotted point represents an average value determined by measurements made upon all specimens of a group. Though shrinkage measurements formed a large part of the test data, being taken on all control specimens, they are here eliminated from consideration, except for Series 5 (Fig. 15) in which there are reinforced columns, for which both time-shrinkage and time-flow curves have been drawn.

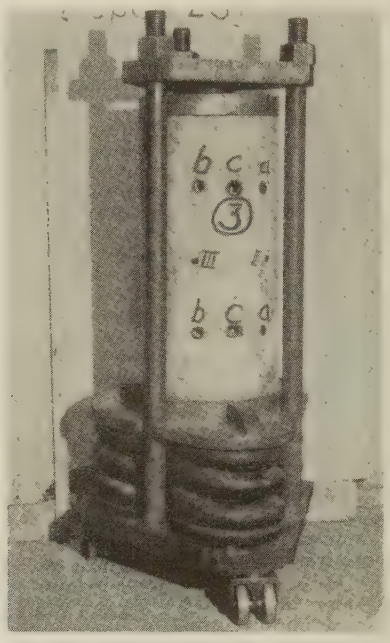


FIG. 3—REINFORCED SPECIMENS UNDER SUSTAINED LOAD, SERIES 5

For the time-flow curves the abscissa represent the time interval in days since the application of load and the ordinates represent flow under the action of sustained load as determined by deducting from the total change in length that due to shrinkage plus that which took place immediately upon application of the load. Examining the figures it will be noted that the flow curves have a characteristic shape, the rate of flow being most rapid immediately after the load is applied and gradually decreasing with time. This indicates that the law governing flow is asymptotic in character, the magnitude of flow gradually approaching a maximum value which depends upon many conditions.

An examination of the diagrams will show that for several conditions the state of flow equilibrium has been reached, but in general it appears that flow is still in progress, even after 3½ years of sustained load.

For several of the figures, in addition to flow curves, there are shown at the left end values of the instantaneous deformations or instantaneous recoveries to the same scale of ordinates as was employed in plotting the flow curves. These deformations and recoveries have been determined by strain-gage observations made immediately before and immediately after the application or release of load.

In Figs. 16 to 19, representing the effect of alternately applying and releasing load, the ordinates to the diagrams give the total deformation due to stress, beginning with the instant before the initial application of the load increment. That is, corrections for shrinkage have been made. The lengths of the full vertical lines represent instantaneous deformations or instantaneous recoveries for the given load increment (760 lb. per sq. in.), i. e., between minimum and maximum or maximum and minimum sustained stresses. The dash lines show the flows under the maximum sustained stress (800 lb. per sq. in.) or the time recoveries under the minimum sustained stress (40 lb. per sq. in.).

Effect Upon Flow of Richness of Mix, Gradation of Aggregate and Water-Cement Ratio—The results of the tests of Series 1 for which the specimens were subjected to a sustained compressive stress of 640 lb. per sq. in. at the age of 7 months, are shown by the diagrams of Fig. 4 and the results of the tests of Series 7, for which the specimens were placed under a sustained stress of 800 lb. per sq. in. at the age of 28 days are likewise represented by the diagrams of Fig. 5. The less precise instruments and methods employed for the earlier series explains the apparent irregularities of the curves of Fig. 4 as compared with those of Fig. 5. Consulting these two figures, it is seen that the general manner in which flow increases with time, as demonstrated by the time-flow curves is characteristically the same for the several groups of specimens represented, the rate of flow being most rapid immediately upon application of the load, and gradually decreasing with time. At the left end of each of the two figures are indicated the magnitudes of the instantaneous deformations which occurred when the load was applied and the instantaneous recovery when the load was released after the period of sustained load which was

approximately 5 months for the specimens of Series 1, and 11 months for the specimens of Series 7.

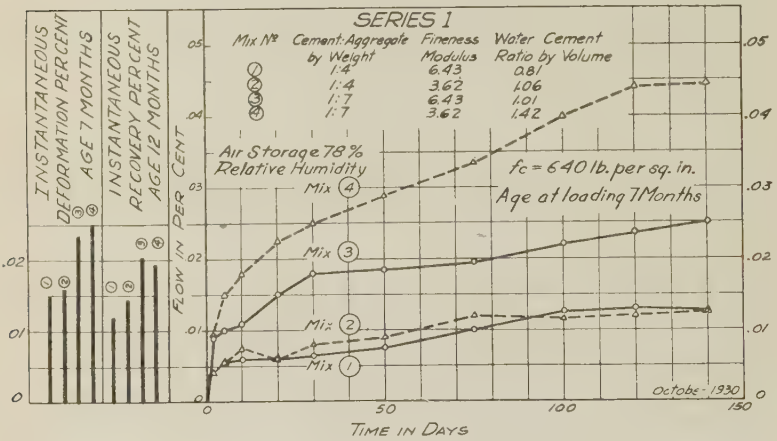


FIG. 4—EFFECT OF RICHNESS OF MIX AND GRADATION OF AGGREGATE UPON FLOW

From a study of the diagrams the following observations are made:

1. Regardless of age, richness of mix, gradation of aggregate, or conditions of storage, concrete subjected to sustained compressive stress plastically deforms for a long time.

2. The rate at which flow takes place under given constant conditions decreases with time, pointing to an eventual state of flow equilibrium.

3. Other things being equal, the leaner the mix, the greater the flow.

4. It appears that the greater the water-cement ratio, other factors remaining constant, the greater the flow, but that the water-cement ratio considered by itself is a lesser factor than is that of the gradation of the aggregate.

5. Other things remaining equal, the better graded the aggregate, or the higher the fineness modulus and the lower the void space, the less the flow.

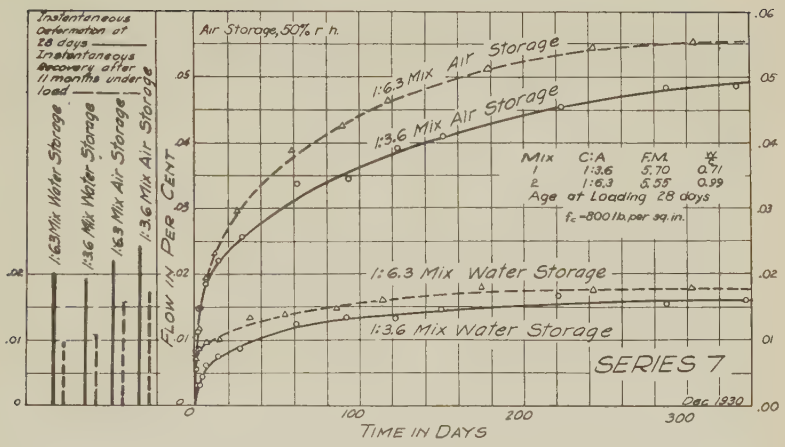


FIG. 5—EFFECT OF RICHNESS OF MIX AND STORAGE CONDITIONS UPON FLOW

6. The instantaneous deformation is less for rich mixes than for lean ones.

7. It is not clear that either the gradation of the aggregate or the water-cement ratio influences the instantaneous deformation to a marked degree.

8. The instantaneous recovery is less than the instantaneous deformation, but this difference in general is not large nor is it clear that the magnitude of the instantaneous recovery is appreciably affected by the duration of the sustained load.

9. Under conditions of dry storage, the flow becomes substantially greater than the instantaneous deformation regardless of richness of mix, gradation of aggregate, or water-cement ratio.

10. It appears that under conditions of dry storage the flow is substantially greater than for the same quality of concrete kept perpetually wet. On the other hand, the instantaneous recovery is proportional to the instantaneous deformation, not being influenced by conditions of storage during the period of sustained load.

11. The total flow at the end of the 5-months sustained load for the lean mix of low fineness modulus of Series 1 (Fig. 4), is more than ½ in. per 100 ft., and for the rich mixes approximately ⅛ in. per 100 ft.

Effect of the Mineral Character of the Aggregate Upon Flow—The results of the tests of Series 4, made to determine the effect of the mineral character of the aggregate are shown by the curves of Fig. 6, one each for limestone, quartz, granite, gravel, basalt, and limestone specimens, and each plotted point representing the average of the given group. The concrete for all aggregates were identical as to richness of mix, water-cement ratio, gradation of aggregate, and conditions of curing. A sustained compressive stress of 800 lb. per sq. in. was applied at the age of 28 days and the specimens were stored in air of 50 per cent relative humidity. The record of flow observations has extended over a period of 2 years.

An examination of the figure shows the following:

12. Regardless of the mineral character of the aggregate, the rate of flow decreases with time, though at the end of two years of sustained load there is no evidence that flow has ceased.

13. The magnitude of the flow is appreciably affected by the character of aggregate, after 2 years being nearly three times as great for sandstone concrete as for limestone concrete, and increasing in the order, limestone, quartz, granite, gravel, basalt, sandstone.

14. As between aggregates of different mineral character, the magnitude of the instantaneous deformation occurring upon application of the sustained load is no criterion to the flow which will eventually take place. The instantaneous deformations for the several aggregates differ but slightly from one another, but the magnitudes of flow differ greatly.

15. The flow after two years of sustained load is 2 to 4 times (depending upon the aggregate) the instantaneous deformation which took place when the load was applied.

16. The rate of flow immediately subsequent to the application of load is generally, though not always, an index to the magnitude of flow after a long period of sustained load. (Among the aggregates investigated, gravel was the exception).

17. The magnitude of the flow produced by the sustained stress of 800 lb. per sq. in. over a period of two years varied from ½ in. per 100 ft. for limestone to 1½ in. per 100 ft. for metamorphic sandstone.

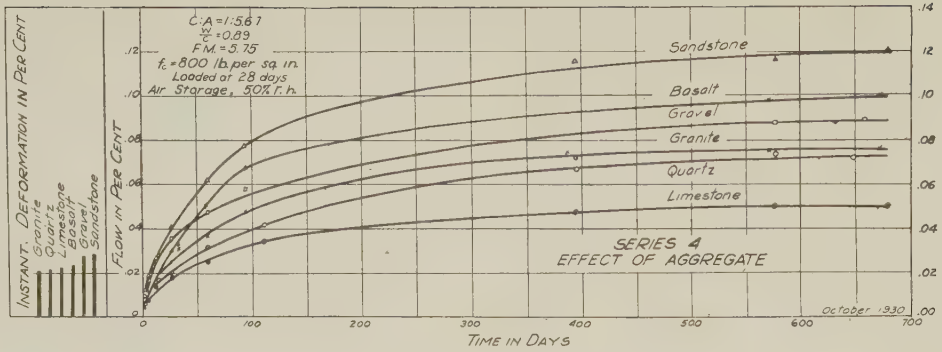


FIG. 6—EFFECT OF MINERAL CHARACTER OF AGGREGATE

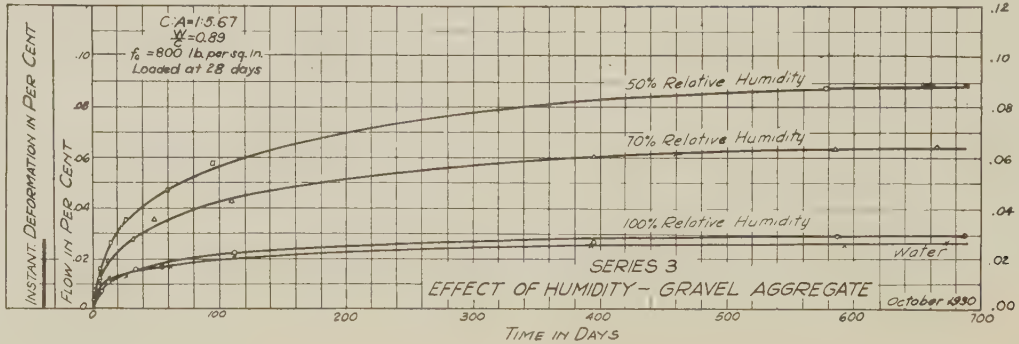


FIG. 8—EFFECT OF MOISTURE CONDITIONS, SERIES 3

Effect of Condition of Storage (As Regards Moisture) Upon Flow—The results of recent tests have demonstrated that the modulus of elasticity of concrete when wet is higher than for the same concrete when dry. In other words, under a given compressive load the instantaneous deformation in a specimen of given concrete saturated with water is less than in a specimen of the same concrete in an air-dry condition, this in spite of the fact that the two specimens may have been moist cured for a long period of time under identical conditions prior to loading.

It has been determined that an analogous condition exists in regard to flow under sustained stress, as is demonstrated by the results of Series 2 and 3, which are shown by Figs. 7 and 8. Further, not only does the analogy hold between fully saturated and air-dry concrete, but also as regards the degree of air-dryness as determined by the humidity of the surrounding air.

Considering Fig. 7, it is seen that there are shown time-flow curves for sustained stresses of 300, 600, 900 and 1200 lb. per sq. in., with ages at loading of 28 days and 3 months, the period of sustained load being about 3½ years. The full lines indicate the flow of groups of specimens stored in air of 70 per cent relative humidity and the dash lines indicate the flow of groups of identical specimens stored under water.

Considering Fig. 8, it is seen that there are shown time-flow curves for nearly two years, a curve for each of four different storage conditions, storage in air at 50 per cent, 70 per cent and 100 per cent (fog) relative humidity and storage under water. For these tests the specimens subjected to a sustained compressive stress of 800 lb. per sq. in. was applied at 28 days. By way of emphasis it perhaps should again be stated that the *flow* is the deformation due to stress alone and does not include shrinkage, which has been eliminated by means of measurements made upon control specimens.

From a study of the diagrams of the two figures the following observations are made:

18. Other things being equal, the flow of concrete stored in air is materially greater than the flow of concrete stored under water.

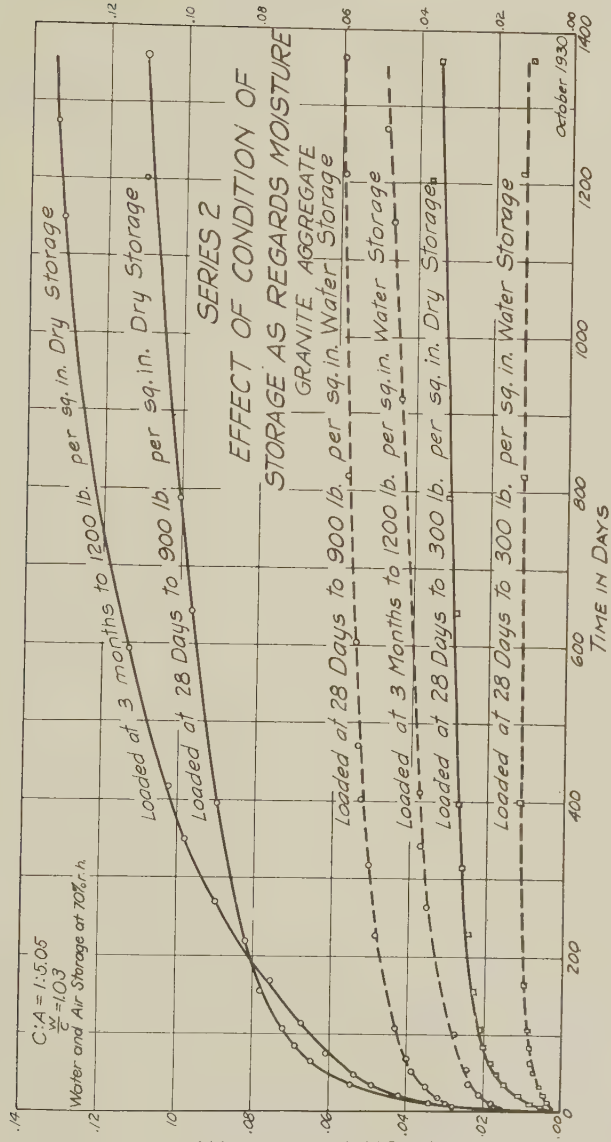


FIG. 7—EFFECT OF MOISTURE CONDITIONS, SERIES 2

and the lower the humidity of the air, the greater the magnitude of the flow. This appears to be true regardless of the intensity or duration of sustained stress and regardless of the age of the concrete at the time of loading.

19. The flows after about 2 years of sustained stress (Fig. 8) at 800 lb. per sq. in. for the several conditions of storage were as follows:

Storage in air at 50 per cent relative humidity	0.089 per cent.
Storage in air at 70 per cent relative humidity	0.063 per cent.
Storage in fog at 100 per cent relative humidity	0.030 per cent.
Storage under water	0.027 per cent.

20. The flow under 800 lb. per sq. in. stress for two years air storage at 50 per cent (Fig. 8) is three times the corresponding flow for concrete stored under water, and three times the instantaneous deformation which took place when the load was applied.

21. The age at loading as well as the intensity of stress and storage conditions as regards moisture has an important influence upon the flow as illustrated by the values of the flow after 3½ years shown on the following table.

TABLE 9—FLOW IN PER CENT AFTER 2½ YEARS, SUSTAINED LOAD—SERIES 2

Storage Condition	Age at Loading, 28 Days			Age at Loading, 3 Months		
	Stress 300 Lbs. per Sq. In.	Stress 600 Lbs. per Sq. In.	Stress 900 Lbs. per Sq. In.	Stress 600 Lbs. per Sq. In.	Stress 900 Lbs. per Sq. In.	Stress 1200 Lbs. per Sq. In.
Air 70% r. h.	0.034	0.069	0.111	0.062	0.081	0.134
Under water	0.012	0.035	0.059	0.018	0.036	0.048

Consulting the table, it is seen that for the range of ages at time loading and the range of sustained stresses represented, the flow in air at 70 per cent relative humidity is 2 to 3 times the corresponding flow under water.

The results of Series 6 and 7 also yield data on the effect of moisture conditions upon the elastic and plastic properties. In Fig. 19 are plotted the residual deformations, due to stress, that remain at the completion of each cycle of certain of the alternating load tests. The maximum stress of each cycle was 800 lb.

per sq. in. and the minimum was 40 lb. per sq. in. A study of the diagrams of this figure results in the following observations:

22. When a sustained compressive stress is alternately applied and released for given periods of time there is left a residual deformation at the end of each stress cycle (no load or low load condition). The magnitude of this residual deformation after a given number of stress cycles is not appreciably influenced by the length of such cycle (high-stress and low-stress periods being of equal duration) but is substantially greater for concrete under conditions of dry air (70 per cent r. h.) storage than under conditions of water storage.

23. After alternations extending over an 8-months period the residual deformation is three times as great for dry as for wet concrete.

In Figs. 20 and 21 are plotted stress-strain diagrams for concrete of two mixes at ages of 5 and 12 months, the ordinates being unit stresses and the abscissae being corresponding unit deformations which took place as the load was gradually applied. The full lines are stress-strain curves for groups of specimens not previously subjected to stress and the dash lines are stress-strain curves for groups which had been previously subjected to a sustained stress of 800 lb. per sq. in. for a period of 5 months for specimens tested at 6 months and for a period of 11 months for specimens tested at 12 months. From these figures it appears that:

24. The ratio of compressive stress to the instantaneous deformation produced thereby, all other conditions remaining the same, is less for a concrete which is dry than for one which is wet (as indicated by the slopes of the curves), regardless of the age of the concrete, the richness of mix, the magnitude of the stress, or whether or not the concrete has been previously subjected to sustained stress.

Effect of Age at Time of Loading Upon Flow—In Fig. 9 are plotted values of flow taking place in the several groups of Series 2 after 3½ years under sustained load. Consulting the figure it is observed that:

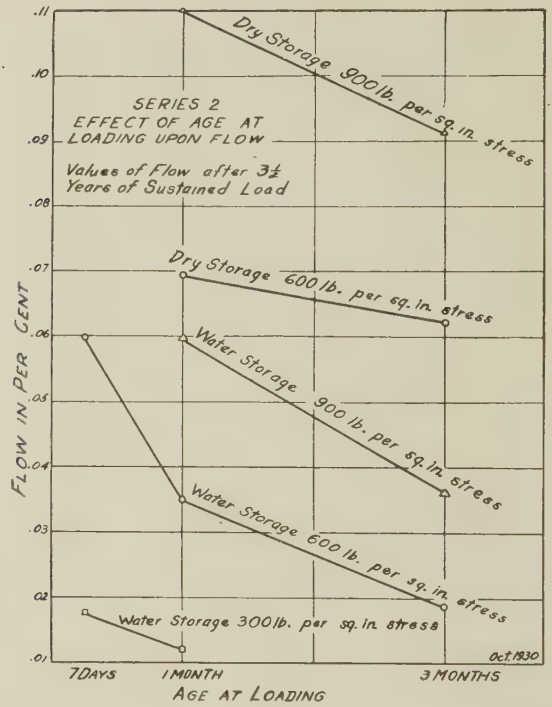


FIG. 9—EFFECT OF AGE OF LOADING UPON FLOW

25. The greater the age at the time of applying the sustained load, the less the flow. Thus the general effect of age is to increase resistance not only to those deformations which take place as the stress is applied but also to the plastic deformations which take place with time.

26. The rate of change in the magnitude of flow under given conditions is greater when the concrete is loaded at an early age than when loaded at a later age.

27. The effect of age at time of loading in reducing the rate of flow is more pronounced for high sustained stresses than for low ones.

Effect of Magnitude of Stress—The effect of the magnitude of the sustained compressive stress upon the flow resulting there-

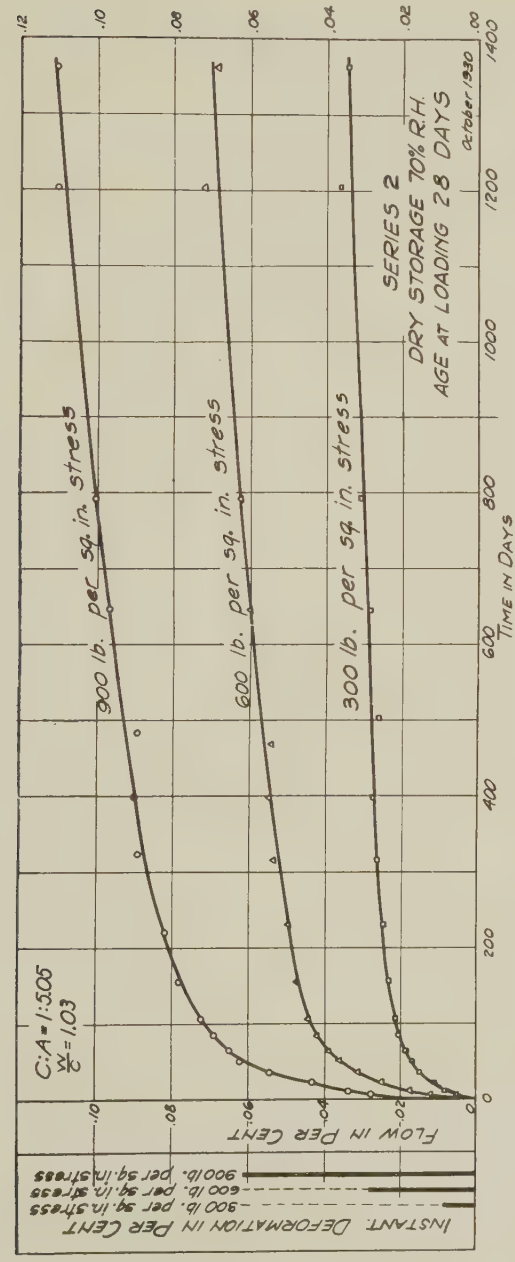


FIG. 10—EFFECT OF STRESS UPON AIR STORED SPECIMENS LOADED AT 28 DAYS—SERIES 2

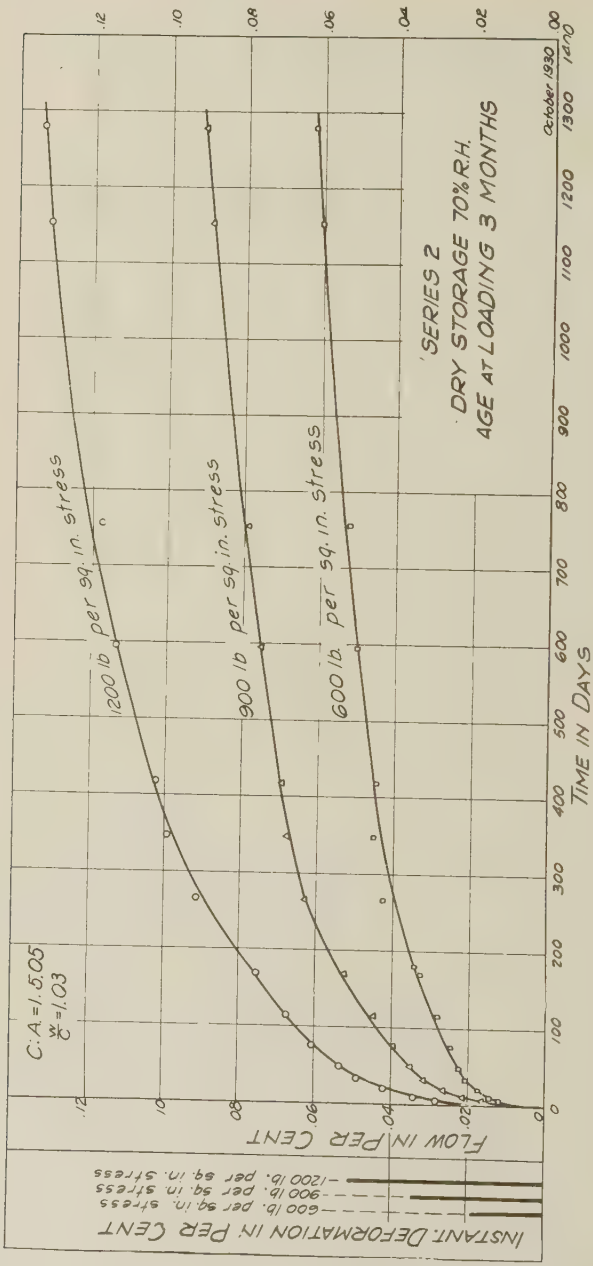


FIG. 11—EFFECT OF STRESS UPON AIR STORED SPECIMENS LOADED AT 3 MONTHS—SERIES 2

from has been determined by the tests of Series 2, where groups of specimens subjected to stresses varying 300 to 1200 lb. per sq. in. have been tested under wet and dry conditions, with loads applied at ages as early as 7 days and as late as 3 months. The results of these tests are shown graphically by the diagrams of Figs. 10 to 14, a separate figure for each condition of storage and age at time of loading. On each figure are shown time-flow curves for the sustained stresses of 300, 600, 900 and 1200 lb. per sq. in., these curves indicating the manner in which flow has taken place over a period of about 3½ years. At the left end of each figure heavy vertical lines show the magnitudes of corresponding instantaneous deformations which took place when the sustained stress was applied.

A study of these four figures results in a number of interesting observations:

28. Under conditions of dry storage, other things being equal, for any given period of sustained load in excess of about four months, the magnitude of the flow proportionally is somewhat greater for high sustained stresses than for low ones, but within the limits of the tests the departure from a linear stress-flow relation is no more marked than is the case for the instantaneous stress-strain ratio or secant modulus of elasticity. It seems reasonable to expect that the sustained stress-flow ratio, as the instantaneous stress-deformation ratio, will become more nearly constant within the range of working stresses as the age is increased.

29. Under conditions of wet storage, including ages of loading of 7 days, 28 days and 3 months, there is marked evidence that the sustained stress-flow relation, at any time after about four months of sustained load, is proportionally greater for high stresses than for low ones, the younger the age of concrete at time of loading, the greater the proportion. Thus by Fig. 12, for an age at loading of 7 days, the flow produced by a stress of 600 lb. per sq. in. is roughly 3½ times that produced by a stress of 300 lb. per sq. in., after any given period of sustained load between 100 and 1300 days. But for a loading age of 3 months (Fig. 14) the corresponding ratio for stresses of 1200 and 600 lb. per sq. in. is roughly 2½.

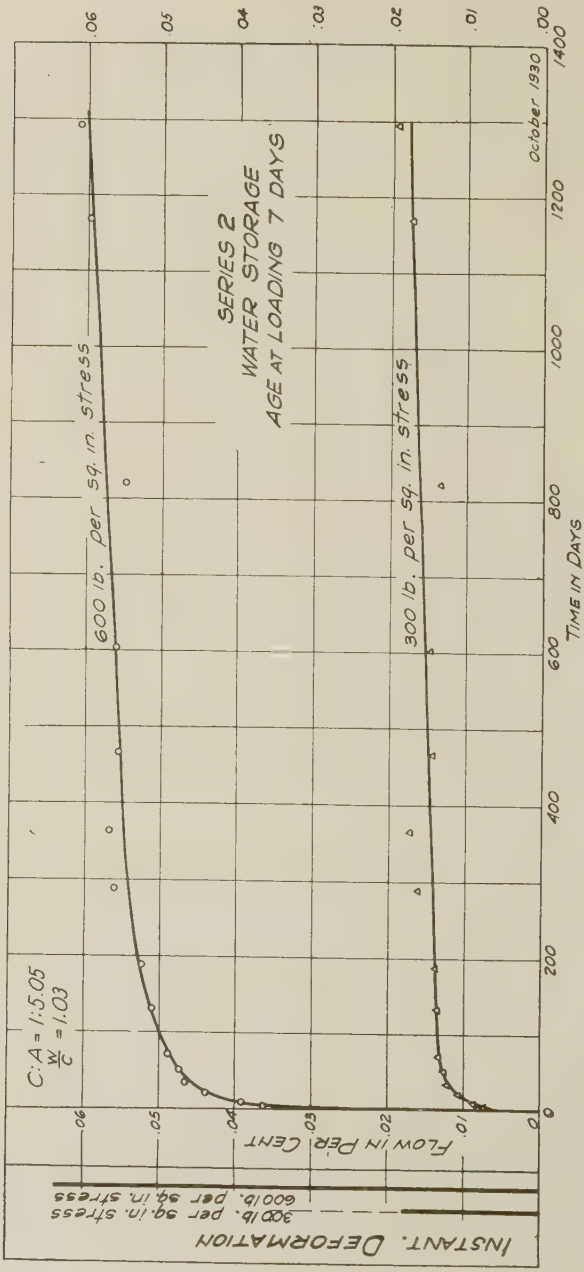


FIG. 12—EFFECT OF STRESS UPON WATER STORED SPECIMENS LOADED AT 7 DAYS—SERIES 2

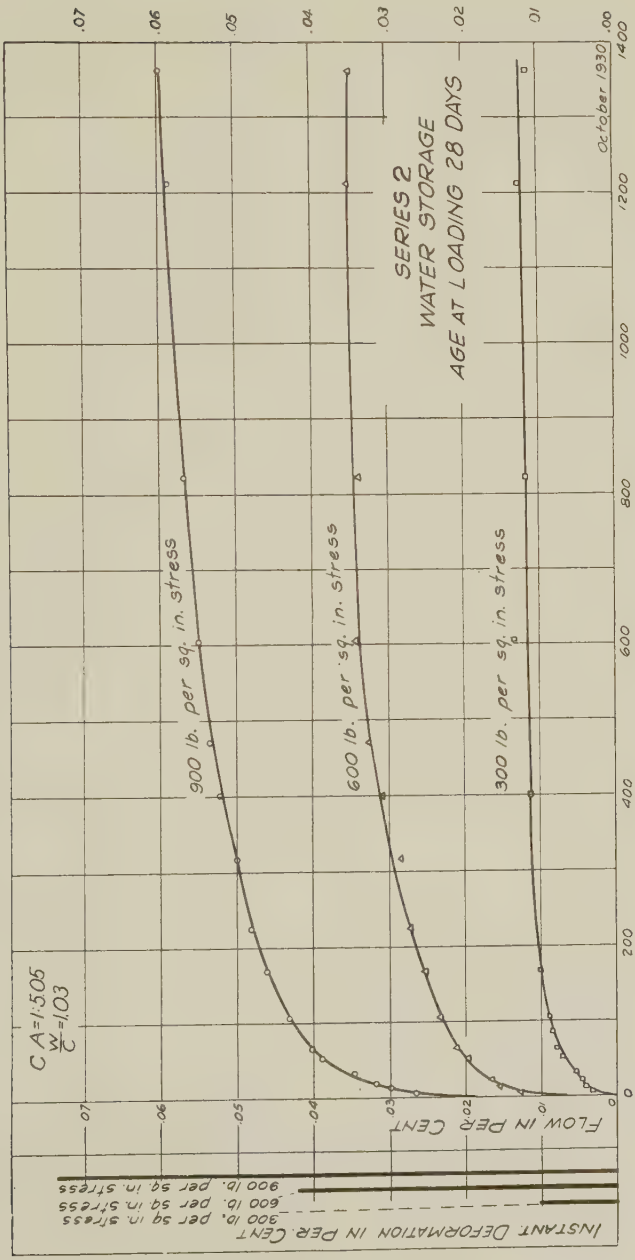


FIG. 13—EFFECT OF STRESS UPON WATER STORED SPECIMENS LOADED AT 28 DAYS—SERIES 2

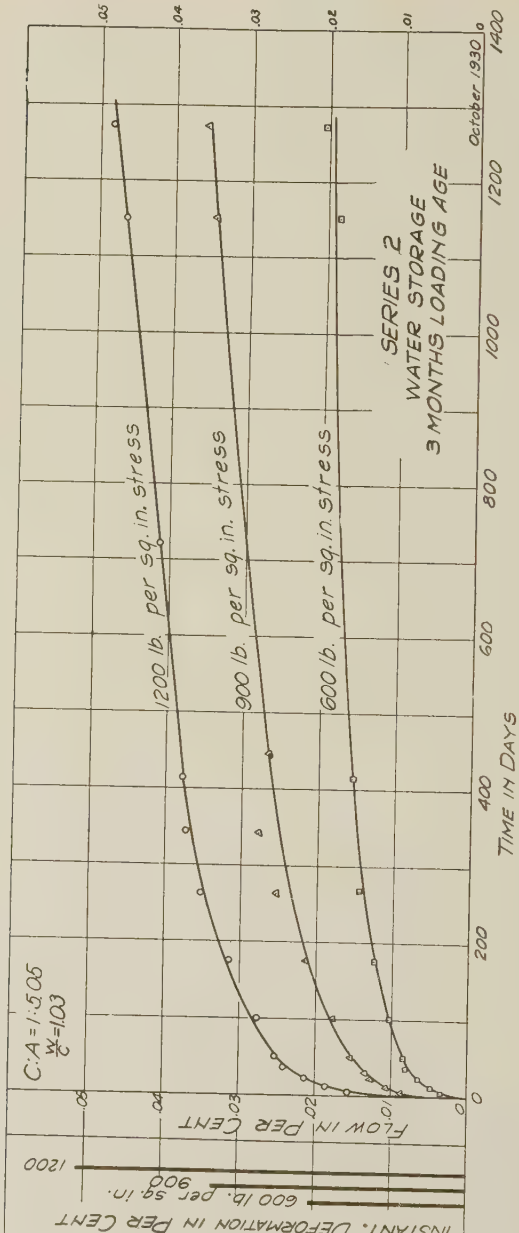


FIG. 14.—EFFECT OF STRESS UPON WATER STORED SPECIMENS LOADED AT 3 MONTHS—SERIES 2

30. For all cases of water storage, regardless of age at time of loading and regardless of intensity of sustained stress, the flow roughly seems to be approaching as a limit the magnitude of the instantaneous deformation. There are some exceptions to this statement, but on the whole, the trend in this direction seems to be rather striking.

31. For all cases of air storage at 70 per cent relative humidity the flow under a sustained stress for 3½ years is substantially greater than the corresponding instantaneous deformation, the ratio of the former to the latter varying between about four for low stresses and about two for high stresses. Since for the higher stresses flow is increasing at a fairly rapid rate even after 3½ years, it would seem reasonable to expect that when flow equilibrium is finally established this ratio of flow to instantaneous deformation may reach a figure as high as four for all stresses. This is, of course, not a general prophecy but merely applies to the conditions of these particular tests.

The data of Series 2 and of Figs. 10 to 14 will be employed in a later discussion of the sustained modulus of resistance, total stress-deformation curves being shown for various periods of sustained loads.

Columns and the Effect of Reinforcement Upon Flow and Stress— The results of the tests of Series 5, made on short plain and reinforced column sections are given by the diagram of Fig. 15, instantaneous deformation being shown at the left and shrinkage and flow curves for plain and reinforced specimens being plotted one above the other.

The tests were begun when the specimens were 60 days old, the previous storage having been 50 days in fog and 10 days in air at 50 per cent relative humidity. The stress was 800 lb. per sq. in. in the plain concrete at the time of loading. The increment of stress placed in the concrete of the reinforced section at the time of loading was 775 lb. per sq. in. (assuming the modulus of elasticity of the steel as 30,000,000 lb. per sq. in.) but the actual stress in both the longitudinal steel and the concrete is not precisely known because observations of shrinkage were not begun upon control specimens until the sustained loads were applied.

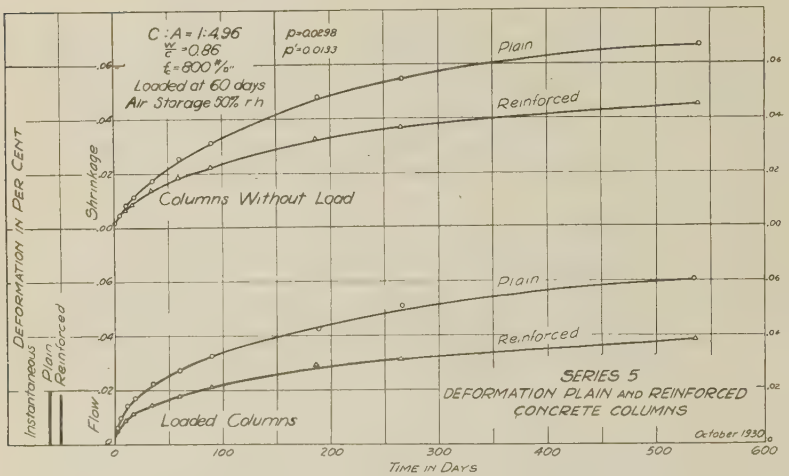


FIG. 15—DEFORMATION OF PLAIN AND REINFORCED CONCRETE COLUMNS—SERIES 5

Considering the curing conditions prior to loading it seems probable that a slight shrinkage had taken place during the 60-days curing period, in which case there was a small initial tension in the concrete and an initial compression in the steel. The longitudinal bars were ground flush with the plane ends of the columns immediately prior to applying the load, and the load was transmitted to the column through heavy steel plates bearing directly against reinforcing steel and concrete; hence stress was transmitted directly from load to reinforcement.

Considering the diagrams of Fig. 15, the following observations are made:

32. The effect of reinforcement is appreciably to reduce both shrinkage and flow.

33. There is a striking similarity between the shrinkage and flow diagrams, both for plain and reinforced specimens, though in general the shrinkage is somewhat greater than the flow.

34. The shrinkage after 18 months in air at 50 per cent relative humidity and the flow produced by a corresponding period of sustained compressive stress are each about ¾ in. per 100 ft. for plain columns and ½ in. per 100 ft. for reinforced columns.

35. For plain columns the combined flow and shrinkage over the loading period of 18 months is approximately six times greater than the instantaneous deformation which took place when the load was first applied.

36. Similarly, for reinforced specimens the combined flow and shrinkage is four times the instantaneous deformation.

37. By computations it can be shown that the compressive stress in the steel produced by load and shrinkage during the 18 months over which the observations extend is made up of the following:

Stress due to instantaneous deformation	5700 lb. per sq. in.
Stress due to flow	11400 lb. per sq. in.
Stress due to shrinkage	13200 lb. per sq. in.
Total change in steel stress	30300 lb. per sq. in.

Since the steel was structural grade, it appears that the stress was near the yield point. Obviously had the load been applied at an early age, as will often be the case in service structures, the yield point would certainly have been reached before 18 months.

38. Assuming the remainder of the load not carried by the longitudinal steel as being uniformly distributed over the full cross-section of the concrete, during the 18 months of sustained load, the sustained stress in the concrete was reduced from 775 lb. per sq. in. to 300 lb. per sq. in.

39. From observations of lateral deformation not here shown, there is computed as existing in the spiral hooping a compressive stress of 13800 lb. per sq. in. for both unloaded and loaded specimens, there being lateral shrinkage but no evidence of an appreciable lateral flow.

40. It seems evident that the yield point of the steel should be a factor in the design of columns carrying a large proportion of dead load when surrounding conditions are such as to produce shrinkage.

Effect of Alternating Periods of Sustained Stress—Figs. 16, 17 and 18 show graphically the results of the tests of Series 6, where specimens were subjected to alternating periods of high and low sustained compressive stress. For this series the specimens were loaded at the age of 28 days, first to a sustained stress of 800 lb. per sq. in. for a given period, then the stress was reduced to 40

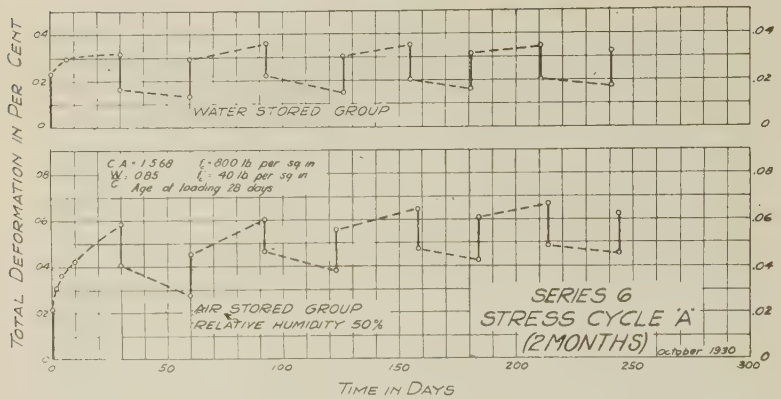


FIG. 16—EFFECT OF STRESS ALTERNATIONS—2 MONTHS CYCLE

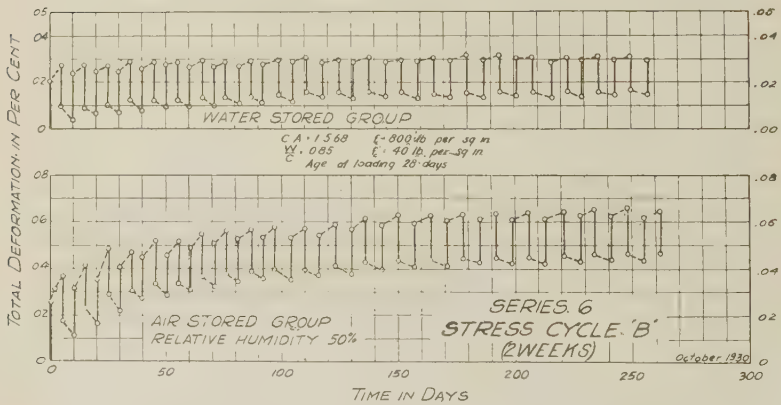


FIG. 17—EFFECT OF STRESS ALTERNATIONS—2 WEEKS CYCLE

lb. per sq. in. for the same length of time, and so the process of alternating high and low sustained stresses was repeated, the length of stress cycle varying between three groups of specimens but remaining substantially constant for the specimens of each group. The length of Stress Cycle A, for which the results are shown by the diagrams of Fig. 16, was two months, that is, each alternation consisted of one month at the stress of 800 lb. per sq. in., followed by one month at 40 lb. per sq. in.; the length of

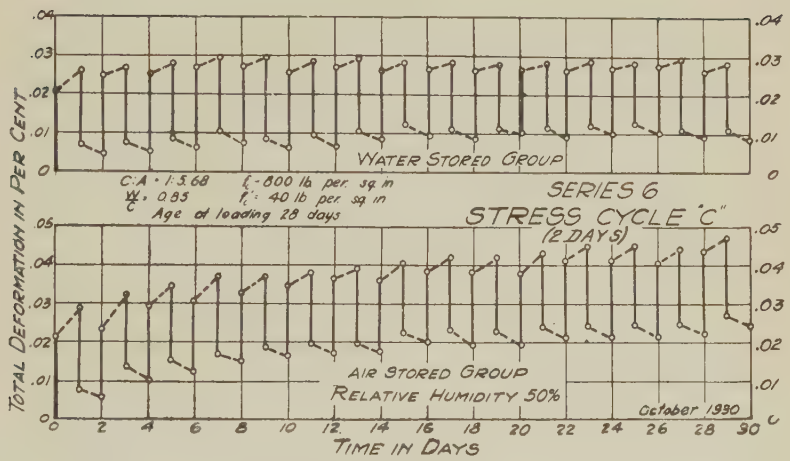


FIG. 18—EFFECT OF STRESS ALTERNATIONS—2 DAYS CYCLE

Stress Cycle B (Fig. 17) at first was 10 days, but later was two weeks; and the length of Stress Cycle C (Fig. 18) was 2 days.

Consulting the three figures, the following significant facts become evident:

41. The instantaneous deformation which took place at the time of first applying the load is greater than any succeeding value of the deformation, and greater than any value of the instantaneous recovery, demonstrating that a permanent set takes place upon the initial application of the load which is not later recovered. This is true regardless of the duration of the stress cycle.

42. Generally speaking, for specimens stored under water, the instantaneous deformations (after the first) and the instantaneous recoveries become somewhat less as the number of stress cycles is increased, but the decrease in these values is not large.

43. As a usual thing, for specimens stored in air there is little if any change in the magnitude of instantaneous deformations (after the first) with the number of alternations of stress, nor are there appreciable variations in the magnitude of the instantaneous recovery with number of alternations.

44. After the initial application of the load, there is general agreement between an instantaneous deformation and the

succeeding instantaneous recovery, regardless of the duration of stress cycle or condition of storage.

45. The rate of flow and of plastic recovery decreases as the number of stress cycles increases.

46. In general, the rate of flow for a given stress cycle is greater than the rate of plastic recovery for the same stress cycle. This characteristic becomes less pronounced as the number of cycles increases; it is more pronounced for specimens in air storage than for those in water storage.

47. Deformations and recoveries, whether instantaneous or plastic, for a given cycle are greater for specimens stored in air than for corresponding specimens stored in water.

Let the "stress deformation" be defined as the decrease from the original length (under minimum load) to that at the middle of any stress cycle at the instant preceding the release of maximum load. Also let "residual deformation" be defined as the decrease from the original length to that at the end of any stress cycle at the instant preceding the application of the maximum load.

48. Both stress deformations and residual deformations increase with the number of stress cycles. There are some indications that the magnitude of the residual deformation continues to increase after the stress deformation has become practically constant.

49. Both stress deformations and residual deformations increase at a more rapid rate with the number of cycles and become substantially larger for specimens in air storage than for those in water storage.

In Fig. 19 are plotted the residual deformations for water-stored and air-stored groups of specimens subjected to 2 months (Cycle A) and 2 weeks (Cycle B) alternations of stress. Consulting the figure it appears that:

50. Under similar stress conditions the residual deformation is approximately the same at the end of a given period of time regardless of the length of the stress cycle. This would also apply to the stress deformations. It is true for specimens in water storage as well as for those in air storage.

51. For Cycles A and B the residual deformations after 8 months of alternating load are approximately 3 times as great

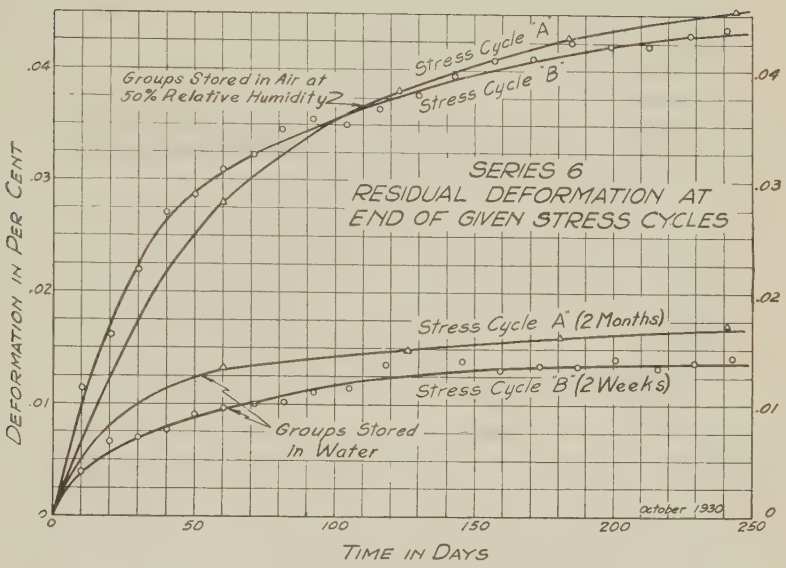


FIG. 19—EFFECT OF ALTERNATING STRESS UPON RESIDUAL DEFORMATIONS

with air-stored as with water-stored specimens; the corresponding stress deformations are twice as great for air-stored as for water-stored specimens.

Effect of Flow Upon Strength and Modulus of Elasticity—Table 10 and Figs. 20 and 21 give the results of tests upon specimens of Series 7 made to determine the effect of flow upon compressive strength and modulus of elasticity. For this series half of the specimens were placed under a sustained compressive stress of 800 lb. per sq. in. at the age of 28 days, while the other half without stress were maintained under corresponding conditions. At the ages of 5 and 12 months, specimens from each of these two groups were tested in compression under a gradually increasing load, the sustained load having been released prior to the application of the ultimate load.

Consulting Table 10 and the diagrams of Figs. 20 and 21, the following facts are in evidence:

52. The effect of flow under the action of a sustained load is to increase the compressive strength over that of identical con-

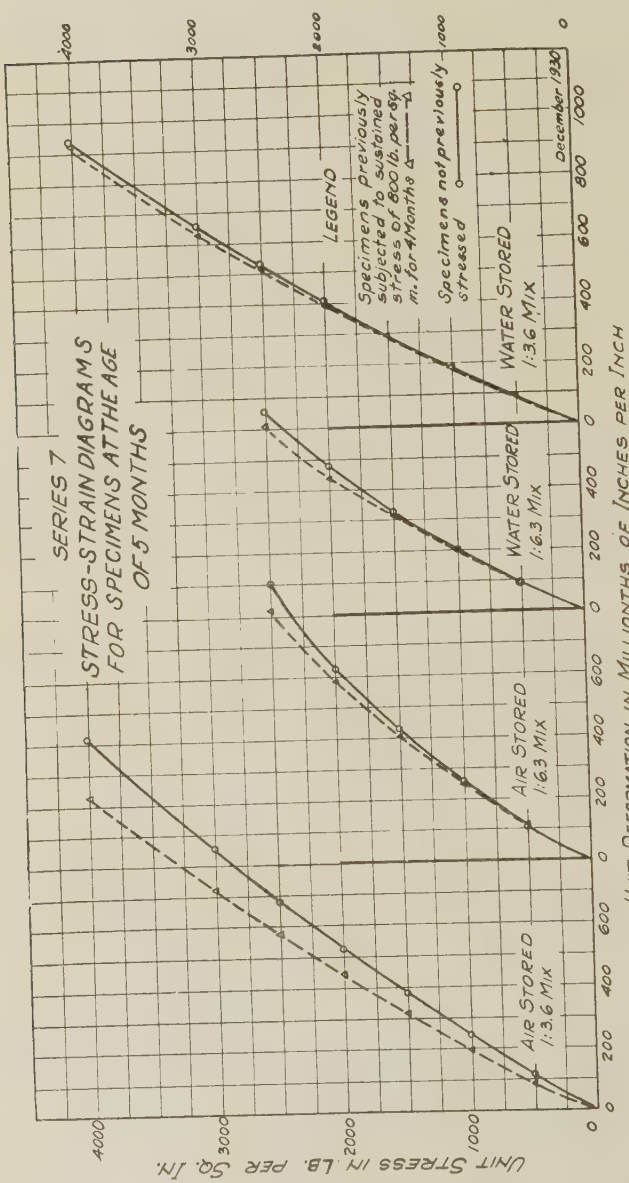


FIG. 20—SERIES 7

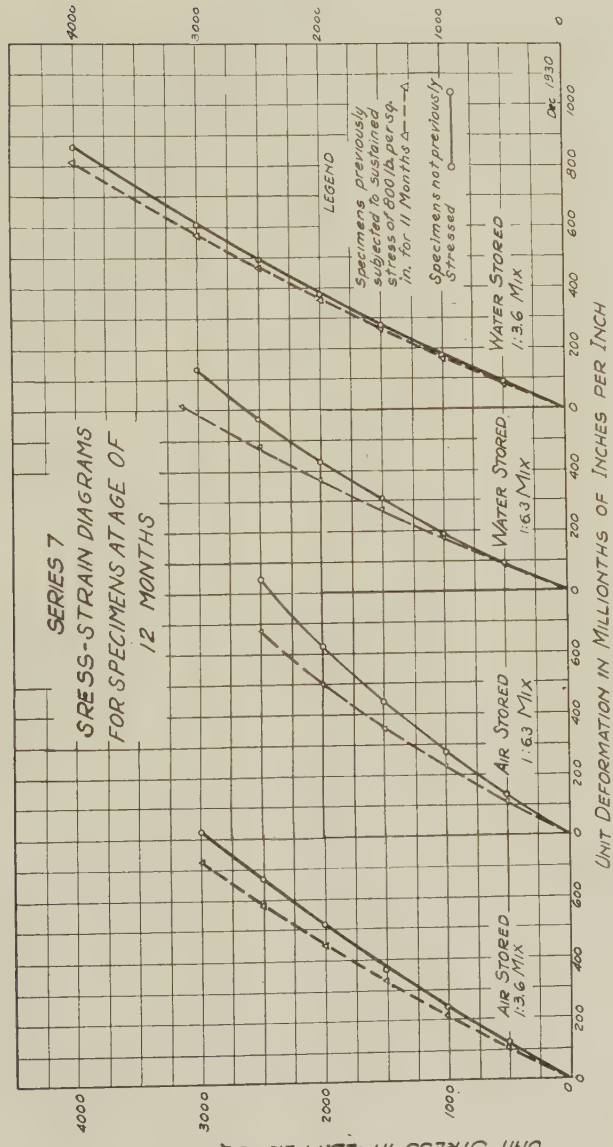


FIG. 21.—SERIES 7

TABLE 10—EFFECT OF FLOW UPON STRENGTH—SERIES 7

Age, Months	Storage	Compressive Strength, Lb. per Sq. In.			
		Rich Mix (1:3.6)		Lean Mix (1:6.3)	
		Stressed	Unstressed	Stressed	Unstressed
5	air	5340	5110	3260	3250
5	water	6220	6020	3460	3560
12	air	5200	4770	3660	3460
12	water	5940	5790	4220	4150

crete not previously stressed, but the amount of the increase in compressive strength due to flow is small. There are some indications that the longer the period of sustained stress, the greater this increase becomes.

53. The influence of moist storage conditions after the age of 28 days upon the compressive strengths at later ages is manifest, since at both 5 and 12 months the water-stored specimens exhibit substantially greater strengths than do corresponding air-stored specimens.

54. The effect of flow under the action of sustained stress is appreciably to increase the instantaneous stress-deformation ratio or the secant modulus of elasticity over that exhibited by identical concrete not previously stressed. This is demonstrated by the curves of Figs. 20 and 21, the dash lines for specimens in which flow had occurred having a substantially greater slope than the full lines for specimens not previously stressed. This is true for high stresses as well as for low ones.

54. The effect of flow is to render the instantaneous stress-deformation relation or modulus of elasticity more nearly constant for all stresses, as indicated by the relative straightness of the corresponding stress-strain diagrams of the figures.

55. The longer the period of sustained load, the greater the increase in the instantaneous modulus of elasticity over that for

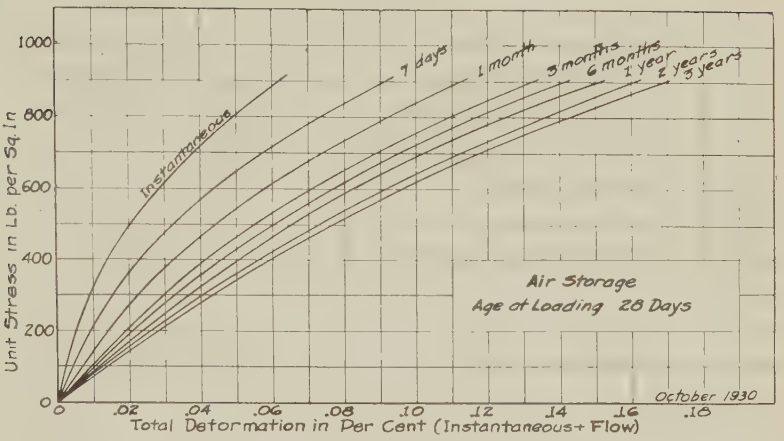


FIG. 22—STRESS-DEFORMATION DIAGRAMS—SERIES 2—AIR STORAGE, LOADED AT 28 DAYS

corresponding specimens not previously stressed. This time effect is more pronounced for the lean mix than for the rich one.

56. The greater the age, the more nearly constant for all stresses becomes the instantaneous modulus of elasticity. This is true for air storage as well as for water storage, and is true for specimens not previously stressed as well as for those previously subjected to sustained load.

57. The instantaneous modulus of elasticity is higher for specimens in wet storage than for corresponding specimens in dry storage. This is true regardless of age, mix, or previous stress condition.

Modulus of Resistance—Perhaps a clearer idea of the effect of sustained compressive stress may be gained by considering the relation between unit sustained stress and total unit deformation, including the instantaneous deformation and flow. With this thought in mind, the diagrams of Figs. 22, 23 and 24 have been prepared from the data of Series 2, the ordinates being unit sustained stresses and the abscissae being unit total deformations. Thus each diagram indicates the stress-strain relation for a group of specimens under the action of stress sustained for the period indicated on the diagram, and these time-stress-deformation curves have been collected into sets, each set for a given condition

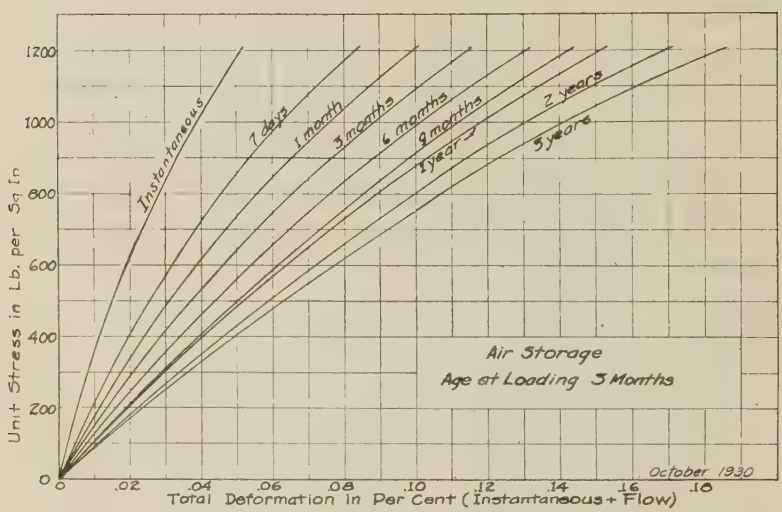


FIG. 23—STRESS-DEFORMATION DIAGRAMS—SERIES 2—AIR STORAGE, LOADED AT 3 MONTHS

of storage and age at application of sustained load. The points employed in plotting these diagrams have been taken from time-flow curves. Hence the curves of Figs. 22-24 pass through plotted points, and for this reason the points are not shown.

Modulus of resistance has been defined as the ratio of unit sustained stress to unit total deformation. Using the diagrams of Figs. 22-24 as a basis, values of the modulus of resistance for various intervals of sustained stress (800 lb. per sq. in.) up to 3 years. The results in millions of pounds per sq. in. are given in Table 11. The "instantaneous" modulus of resistance is merely the secant modulus of elasticity in the usual terminology.

Similarly computed values of the modulus of resistance for Series 3 and 4 are given in Table 12.

A study of the diagrams and tables indicates that the following relations obtain:

58. The modulus of resistance for concrete under constant sustained compressive stress decreases with time, at first rapidly and then more and more gradually for a long period. Under normal working stresses the sustained modulus of resistance

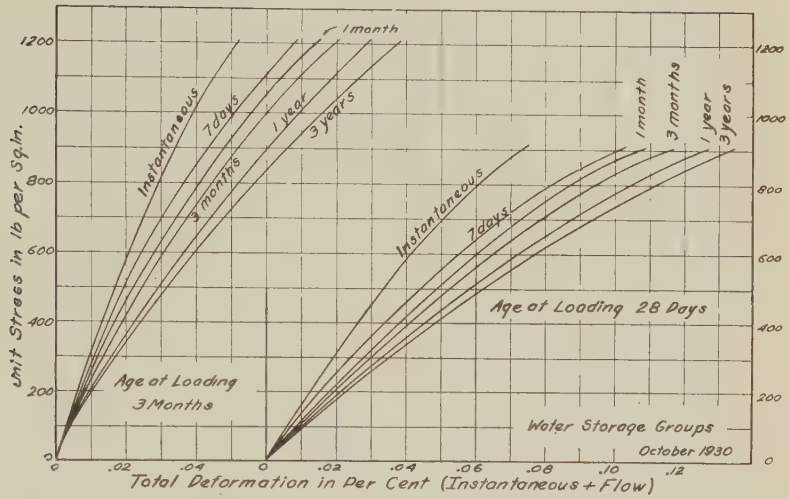


FIG. 24—STRESS-DEFORMATION DIAGRAMS—SERIES 2—WATER STORAGE, LOADED AT 28 DAYS AND 3 MONTHS

TABLE 11—VARIATION OF MODULUS OF RESISTANCE WITH PERIOD OF SUSTAINED LOAD—SERIES 2

Period of Sustained Stress	Modulus of Resistance in Millions of Lb. per Sq. In.*			
	Air Storage		Water Storage	
	Age at Loading, 28 Days	Age at Loading, 3 Months	Age at Loading, 28 Days	Age at Loading, 3 Months
Instantaneous	1.63	2.72	1.26	2.76
7 Days	1.09	1.64	0.93	2.20
30 Days	0.87	1.37	0.86	1.95
3 Months	0.73	1.17	0.81	1.81
6 Months	0.68	1.01	....	....
1 Year	0.64	0.74	0.74	1.53
2 Years	0.59	0.67	0.71	1.48
3 Years	0.56	0.62	0.70	1.42

*At a stress of 800 lb. per sq. in.

eventually becomes but a fractional part of the instantaneous value that was obtained when the load was applied. (This fractional part under certain conditions was less than one fifth.)

TABLE 12—VARIATION OF MODULUS OF RESISTANCE WITH PERIOD OF SUSTAINED STRESS—SERIES 3 AND 4

Period of Sustained Stress	Modulus of Resistance in Millions of Lb. per Sq. In.*								
	Gravel				Sand-stone	Basalt	Gran-ite	Quartz	Lime-stone
	50% r. h.	70% r. h.	100% r. h.	Water	50% r. h.	50% r. h.	50% r. h.	50% r. h.	50% r. h.
Instant.	2.91	2.91	2.91	2.91	2.81	3.56	3.90	3.72	3.56
1 Month	1.26	1.47	1.93	1.93	1.13	1.24	1.70	1.84	1.93
3 Months	0.97	1.17	1.65	1.72	0.76	0.90	1.20	1.37	1.48
6 Months	0.84	1.03	1.54	1.62	0.65	0.79	0.99	1.09	1.25
9 Months	0.75	0.96	1.48	1.55	0.61	0.73	0.90	0.98	1.20
12 Months	0.73	0.92	1.44	1.52	0.58	0.70	0.86	0.91	1.15
18 Months	0.70	0.88	1.42	1.47	0.55	0.67	0.83	0.87	1.11

*At a stress of 800 lb. per sq. in.

59. Under like conditions the modulus of resistance of concrete loaded at an early age always remains less than the corresponding modulus for the same quality of concrete loaded at a later age.

60. Other things remaining equal, the conditions of storage as regards moisture have an important bearing upon the magnitude of the modulus of resistance. Regardless of the period of sustained load, the sustained modulus of resistance is higher for a concrete in water storage than in air storage and is higher for a concrete surrounded by air of high humidity than for one surrounded by air of low humidity.

61. The mineral character of the aggregate influences to a marked degree not only the instantaneous modulus of elasticity, but even more so the sustained modulus of resistance. After 18 months of sustained load (Table 12) the modulus for limestone concrete is double that for sandstone concrete. The corresponding ratio for the instantaneous modulus is $\frac{4}{3}$.

Plastic Recovery—In considering the effect of alternating loads the question of plastic recovery received some attention. It was shown by diagrams that following each release of load there was an instantaneous recovery and that this was followed by a plastic recovery which increased with time.

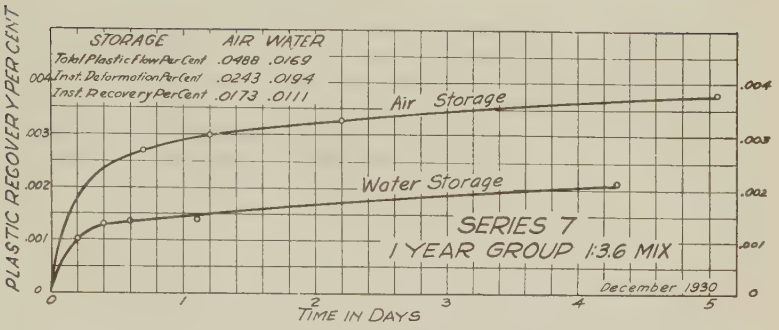


FIG. 25—PLASTIC RECOVERY—SERIES 7

The manner in which this plastic recovery takes place, and its magnitude when plastic equilibrium has been established, as compared with the preceding flow, are of interest.

Fig. 25 has been prepared by plotting observations of plastic recovery made on specimens of Series 7 which were tested in compression at the age of one year, after having been under load for 11 months. After the sustained load was released the specimens of this group were observed at intervals for several days in order to determine the general shape of the recovery curves for both air and water-storage.

Similarly, Fig. 26 shows residual deformation and plastic recovery curves for Group C of the alternating load tests of Series 6. The specimens of this group after fifteen 2-day alternations of stress were allowed to rest for a period of about 7 months, observations of changes of length being made periodically for both wet and dry groups. The full line curves of Fig. 26 are drawn through points which represent the residual deformations which were measured at the end of each successive 2-day cycle of stress, the measurement having been made just before another application of the load. The dash lines indicate the plastic recovery during the 7 months following the release of the last alternating load during which period the specimens were at rest.

An examination of Figs. 25 and 26 shows that:

62. The general shape of the time-recovery curves is much the same as for corresponding time-flow curves, but the rate of

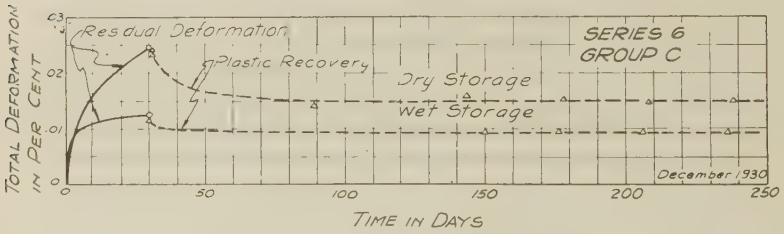


FIG. 26—PLASTIC RECOVERY—SERIES 6, GROUP C

recovery is comparatively small, and the state of plastic equilibrium is reached in a comparatively short time.

63. Plastic recovery for concrete in water storage is considerably less than the corresponding plastic recovery in air storage.

64. The plastic recovery as compared with the flow is a comparatively small quantity, much of it taking place within a few hours after the release of load. For example, Fig. 25 shows that the plastic recovery in 1 day is ¾ that in 5 days. Fig. 26 indicates that there is no further recovery after perhaps 2 months. Also, from Fig. 25 the plastic recoveries are respectively 0.004 and 0.002 for the dry and wet groups; while the corresponding flows for these groups, due to previously sustained load, were 0.048 and 0.016 per cent, values roughly ten times as great.

Summary of Flow Values—In order to render easy a comparison of the principal data concerning deformations produced by sustained stress which have been presented in the form of tables and graphs in the preceding pages, Table 13 has been prepared. In the Column headed "Instant Deformation," are given, in per cent, values of the deformations which took place immediately upon application of the sustained compressive stress stated under "Conditions." In the last column are recorded the plastic deformations or flows which were produced by the sustained stress subsequent to its application and during the time interval stated under "Conditions."

TABLE 13—SUMMARY OF OBSERVED DEFORMATIONS

Conditions	Instant. Deform. Per Cent	Flow Per Cent
Series 2—Period of Sustained Stress—1300 Days		
Dry storage, 28-day age at loading,		
Air at 70% relative humidity.		
Stress 900 lb. per sq. in.	0.06	0.11
600 do	0.03	0.07
300 do	0.01	0.03
Dry storage, 3 months age at loading		
Stress 1200 lb. per sq. in.	0.05	0.13
900 do	0.04	0.09
600 do	0.02	0.06
Water storage, 28 days age at loading		
Stress 900 lb. per sq. in.	0.07	0.06
600 do	0.04	0.04
300 do	0.01	0.01
Water storage, 3 months age at loading		
Stress 1200 lb. per sq. in.	0.05	0.05
900 do	0.03	0.04
600 do	0.02	0.02
Water storage, 7 day age at loading		
Stress 600 lb. per sq. in.	0.06	0.06
300 do	0.02	0.02
Series 3—Period of Sustained Stress—650 Days		
Sustained stress 800 lb. per sq. in.		
Air at 50% relative humidity	0.03	0.09
70% do	0.03	0.06
100% do	0.03	0.03
Water	0.03	0.03
Series 4—Period of Sustained Stress—650 Days		
Air at 50% relative humidity		
Stress 800 lb. per sq. in.		
Sandstone aggregate	0.03	0.12
Basalt aggregate	0.02	0.10
Granite aggregate	0.02	0.08
Quartz aggregate	0.02	0.07
Limestone aggregate	0.02	0.05
Series 5—Period of Sustained Stress—500 Days		
Stress in concrete at time of loading, 800 lb. per sq. in.		
Age at loading, 2 months.		
Storage 50% relative humidity		
Plain	0.02	0.06
Reinforced	0.02	0.04
Series 7—Period of Sustained Stress—300 Days		
Air at 50% relative humidity		
1:6.3 mix	0.02	0.06
1:3.6 mix	0.02	0.04
Water		
1:6.3 mix	0.02	0.02
1:3.6 mix	0.02	0.02

PART 2—SUMMARY OF VARIOUS PUBLISHED DATA CONCERNING FLOW OF CONCRETE

Following are brief abstracts of what are regarded as some of the more important published reports describing tests for which the phenomenon of gradually increasing deformations under the action of sustained loads has been observed and the results have been discussed. These include laboratory tests upon specimens, in the form of reinforced beams and slabs subjected to transverse loading and cylinders subjected to axial compressive stress, as well as field observations upon beams, slabs and columns in service structures in which deformations in both concrete and steel were measured over varying periods of time.

Among these published accounts there is no uniformity in the use of a term for the deformation due to sustained stress, it being variously designated as "plastic flow," "time yield," "time deformation," "flow," etc., and in the majority of tests reported the factor of shrinkage has been combined with that of flow without attempt at separation, their sum being considered as the "time yield." Obviously, "flow" is primarily a function of magnitude of stress, while "shrinkage" is primarily influenced by the surrounding moisture conditions, which determine the rate at which the concrete is relieved of its uncombined moisture.

At the end appears a summary of conclusions and a bibliography of articles, papers and discussions pertaining to flow and shrinkage.

Notes on the Effect of the Time Element in Loading Reinforced Concrete Beams, by W. K. Hatt, Proc. Am. Soc. T. M., vol. 7, 1907, p. 41. The tests were made during 1905 to 1907 at the Laboratory for Testing Materials, Purdue University, under the following loading conditions:

1. Continuously increased until failure.
2. Repeated loads.
3. Sustained loads.

The beams were of 1:2:4 broken stone concrete, 8 in. wide by 10 in. to the depth of the steel. They were loaded at the one-third points, the span length being 9 ft. except for the sustained load tests for which the span varied from 8 to 12 ft. in order to obtain various desired stresses in the steel. The percentage of steel was 1.00 except for the sustained load tests, where the percentage varies from 0.75 to 1.50 for the reason stated above. Age at test was about 60 days. It appears that the beams were stored in open air, without tempera-

ture control and without temperature or humidity records, and no account is taken of shrinkage due to moisture change. Following are tabulated the important results, average values being given.

DEFLECTION OF REINFORCED CONCRETE BEAMS (HATT)
Values in Inches

Approx. Stress in Steel Lb. per Sq. In.	Ordinary Load, Testing Machine	Repeated Load			Sustained Load, 2 Mo.
		Times Applied	Deflection	Set	
3000	0.050	1 710	0.025 0.035	0.005 0.005	0.100
8000	0.080	1 500	0.070 0.155	0.010 0.055	0.160
16000	0.140	1 515	0.195 0.250	0.060 0.090	0.290
29700	0.210	1 550	0.285 0.300	0.095	0.360

INCREASE IN DEFLECTION UNDER SUSTAINED LOAD (HATT)

Initial Stress, Lb. per Sq. In.	First Recorded Deflection, In.	Deflection after 1½ Months, In.
3000	0.04	0.10
8000	0.10	0.17
16000	0.15	0.29
29700	0.20	0.36

In his discussion Hatt says, "These results, taken together, show a sort of plasticity in concrete, by which it yields under the action of a load applied for a long time, or applied a number of times."

While the conditions surrounding the tests are not sufficiently well established to make the quantitative data of much value, these tests made nearly a quarter century ago clearly demonstrated that the time yield or plastic flow of concrete under the action of sustained flexural stresses was a quantity of considerable magnitude.

Shrinkage and Time Effects in Reinforced Concrete, by F. R. McMillan, Univ. of Minn., Studies in Eng. Bull. 3, 1915.

The tests here reported were made at the University of Minnesota in the years 1913-1914 under the direction of F. R. McMillan. They comprised observations on several beams and slabs (stored in air) which were subjected to sustained load. Deformations were measured between points set in the concrete in the top of the beams or slabs and in the steel in the bottom. Center deflections were also observed. The temperature ranged between 60 deg. and 80 deg. F. No record was kept of the humidity. The object of the tests was

to study shrinkage, plastic flow (called time effect in the paper) and Poisson's Ratio.

Test of a 30-in. by 5½-in. by 12-ft. beam.—The concrete was a 1:2:5 mix, the coarse aggregate being crushed limestone. 8 by 6-in. test cylinders made from the same concrete gave at 27 days an average compressive strength of 2500 lb. per sq. in. and a modulus of elasticity at f_c = 1000 of 3,500,000 lb. per sq. in.; 4 by 5-in. transverse specimens gave an average modulus of rupture of about 500 lb. per sq. in. The steel ratio for the longitudinal was 0.00796 at a depth of 4⅝ in. from the top of the beam. The transverse steel was ⅜-in. plain round rods spaced 12 in. on centers at a depth of 5 in. No curing data are given except the statement that the beam was left exposed to the laboratory air. At the age of 41 days the beam was placed on supports thus applying a dead load calculated to produce a maximum steel stress of 8000 lbs. per sq. in., and a concrete stress of 335 lb. per sq. in. was applied. At 80 days a live load (sand-bags) of 500 lb. was applied, effective at the third points. After sundry applications and removals of load, at the age of 107 days the full live load of 1500 lb. calculated to give the same maximum stress as the dead load, was applied and sustained except for periodic instantaneous removal and reapplication.

The effect of shrinkage was determined by transverse measurements and these corrections have been applied to the longitudinal deformation given.

The actual stresses for a live load change calculated from the observed deformations, assuming a modulus of elasticity of 30,000,000 for the steel and 3,500,000 for the concrete, are about 300 lb. per sq. in. for the concrete and about 2600 lb. per sq. in. for the steel. The deformation due to stress in the concrete at the center of span after almost 60 days of dead load, was about twice the instantaneous deformation due to dead load. After 600 days, the total deformation due to stress was about three times the sum of the instantaneous dead and live load deformations. At the end of this period, the total deformation at the center of the span, due to stress, was about 0.11 per cent of the gage length. Including the shrinkage, the total deformation after 600 days was about four times the instantaneous deformation. The observed deformation in the steel at the end of a 530-day period which began with the application of the live load, which deformation corresponds to a steel stress of 12,000 lb. per sq. in., was about five times the instantaneous live load deformation. The effect of shrinkage before cracking or bond failure occurs is to decrease the observed steel deformation.

Tests of four 4 by 5 by 42-in. beams.—This group of tests was undertaken to study further the time effect and shrinkage in concrete. Four similar beams were made and stored under the same conditions for the entire period of test. On one which was without load, the shrinkage alone was measured, while on the others, the flow due to sustained load over a period of time was observed.

The concrete was 1:2:4 mix, the coarse aggregate being crushed limestone of pea size. The specimens, which were 4 by 5 by 48-in. beams, were covered with wet sacks at the time of casting, but were left open to the air after the

first day. The depth to steel was 4½-in. The steel ratio was 0.0078 for the first beam and 0.0085 for the others.

Each beam was tested on a 42-in. span with a center load of 860 lb., calculated to give approximately 650 lb. per sq. in. stress in the concrete and 15,000 lb. per sq. in. stress in the steel. Measurements of longitudinal deformation were made both on the top of the beam and in the steel, and center deflections were observed.

It appears that there was considerable variation in humidity conditions with a considerable resulting effect on the regularity of shrinkage and with some effect on flow. The total deformations due to load alone after a year were from 0.08 to 0.12 per cent, which values were from 3 to 4 times the instantaneous deformation.

Slab tests.—A 6 by 8-ft. by 3-in. thick two-way slab, supported at four edges, and a 10 by 10-ft. by 4¾-in. thick one-way slab, supported at two edges, were also tested under the action of sustained load.

For the greater part of the time, the live load on the 6 by 8-ft. slab was 100 lb. per sq. ft. Only the dead load of its own weight was applied to the 10 by 10-ft. slab.

Deformation and deflection measurements were made as in the beam tests, except that the shrinkage observations were made on the edges of the slabs over the supports.

Much the same results as were obtained in the beam tests as regards time deformation, time-deflection trends, and ratio of total load deformation to instantaneous load deformation, were obtained. Two noteworthy results manifested themselves. Due to shrinkage alone, the steel of both one-way and two-way slabs was under considerable compression, this shrinkage producing a deflection which continued to increase as the shrinkage increased; and in the two-way slab, the yielding of the concrete due to sustained stress was a higher multiple of the instantaneous load-deformation in the transverse direction than in the longitudinal direction, showing that the rate of yielding increased with stress.

Significance.—These tests served not only to demonstrate that concrete does yield considerably with time, but also point to the probable effects of flow, especially when combined with shrinkage, on compound structural members, such as reinforced concrete beams and slabs.

In tests of beams and slabs, with the zero of live load measurements established under dead load conditions of no cracks and the steel under compression due to shrinkage, if cracks are produced by the application of live load, there would be thrown on to the steel the dead load tension previously carried by the concrete and thus the apparent deformation due to live load would be greater than its true value. Also the gradual increase of steel stress with time may be due either to further yielding of the concrete in tension, thus opening up new cracks, or to a complete breaking down of the bond through the region of maximum moment.

In members subjected to flexure, the continued deflection is augmented by shrinkage, since the compression fibers, in addition to yielding plastically,

must shrink a greater amount than those on the tension side because of the restraining effect of the steel. Furthermore, the continued shrinkage on the tension side operates to increase the number of cracks and thus increase the average deformation, and hence the deflection.

In regard to the slab tests, it was found that under the condition of full dead and live load stress, the tensile steel at center of span showed compression after a year of test. It is possible that in a service structure, where a slab is not free to shrink in all directions, this condition would not occur.

The test of the 10 by 10-ft. slab showed that the time yield had a much greater effect on the continued deflection than did the shrinkage.

Thus it appears that in determining the probable total actual stress in any flexural member after any period of time, flow, coupled with shrinkage is a factor for consideration.

Time Tests of Concrete, by A. H. Fuller and C. C. More, Proc. Am. Conc. Inst., vol. 12, 1916, p. 302.

This paper reports tests made in 1915 at the University of Washington in connection with measurements of deformation in a reinforced concrete building, as result of observations that deformations in the building increased with time as the load remained constant. The test consisted of first applying load to 3 compression specimens (cast from the same concrete as, or cut from, the building), according to a schedule which was approximately the same as the loading schedule for the building, after which treatment the load was allowed to remain constant. The time from the beginning of the first part of the test to the beginning of the sustained load test, is different for each specimen. No information is given regarding the character of the concrete and the curing and environmental conditions of the building and specimens, nor is it stated whether the observations in the building were on compression or flexural members. Shrinkage was included in the values of deformation.

The thesis of the authors is stated as follows: "The modulus of elasticity is a measure of the relation of stress to deformation as determined from time tests in which the rate of application is the same (as in a structure on which time-load observations were being made) would furnish means for making a fairly satisfactory determination of the existing stresses." This would appear to be a satisfactory method of approximately determining the stresses in a structural member under observation provided only that all the influencing conditions are the same for the specimen as for the structure.

For the sustained load tests, the data which are given by the investigators represent values of deformation measured from the start of the previous loading operations and which values therefore include what flow occurred during the previous test. A comparison of the rates of flow, however, is instructive and in the following table are shown the flow between the 25th and the 60th day after the beginning of the test.

From this tabulation it will be noted that the rate of flow is greater for higher stresses than for lower stresses.

A discussion of this paper by F. R. McMillan presents data in diagramatic form illustrating the use of the above mentioned method in determining the

INCREASE IN DEFORMATION IN PER CENT
25 to 60 Days After Zero of Test

Specimen	Stress, Lb. per Sq. In.	Increment of Deformation, %
A	650	0.064
B	650	0.060
C	1150	0.090

probable actual stresses in a structure. Cases of columns in a service structure are also cited, in which it is shown that some 200 days after measurements were begun an increase in steel stress due to combined flow and shrinkage amounted in one case to as much as 22,000 lb. per sq. in.

The Flow of Concrete under Sustained Load, by E. B. Smith, Proc. Am. Conc. Inst., vol. 12, 1916, p. 317.

In 1914-1915, E. B. Smith, then with U. S. Bureau of Public Roads, Washington, D. C., conducted flow tests on a reinforced concrete beam and on concrete compression cylinders.

For the flexural tests there was employed a gravel concrete beam of 1:2:4 mix; reinforcement, approximately 0.75 per cent; dimensions 5 in. wide, 8 in. full depth, 7 in. effective depth, clear span 10 ft.; loaded at center with concentrated load of 1225 lb., calculated to produce a maximum stress of 1000 lb. per sq. in.; loaded at the age of two weeks, strain measurements along top fibers of beam. Load was sustained for 19 days, when it was released and readings were continued for 9 days more. Storage conditions are not stated.

Test results show:

1. Deformation increased with length of period of sustained load (including both shrinkage and flow).
2. A maximum deformation at end of 19 days for the top fibers of the beam at the center of span was 0.035 per cent.
3. Maximum deformation was 2½ to 3 times the instantaneous deformation.
4. Instantaneous deformation was greater than instantaneous recovery after 19 days of load.
5. Plastic recovery very small.

The compression tests were made on two 6 by 24-in. cylinders of a 1:2:4 gravel concrete loaded to a stress of 700 lb. per sq. in. One was stored in air and one was kept saturated with water. Measurements of longitudinal deformations were made on a 20-in. gage length. Corresponding unstressed cylinders were maintained under parallel storage conditions, in order that dimensional changes due to causes other than stress might be determined. All cylinders were cured for 29 days in wet sand and the dry storage specimens were thereafter kept 7 days more in air before loading.

The wet specimen was unloaded after 17 days under load, at which time the flow (making corrections for shrinkage) was 0.036 per cent. The dry specimen

was unloaded after 28 days of sustained load, the flow being 0.046 per cent. After 17 days under load, the flow in the wet specimen was about 82 per cent of that of the dry specimen.

Tests of Large Reinforced Concrete Slabs, by A. T. Goldbeck and E. B. Smith, Proc. Am. Conc. Inst., vol. 12, 1916, p. 324.

During the period 1914-16, the authors of the above named paper conducted at the U. S. Bureau of Public Roads, Washington, D. C., tests on reinforced concrete slabs, during which tests a time-yield was noticed. In part of their discussion they offer the following evidence which is not supported by any attendant data:

"During the tests of one of these slabs . . . this phenomenon was first discovered and a series of experiments was begun to obtain data regarding the flow of concrete. Since it is not the object of this paper to discuss the facts or theory . . . the following statement . . . (is given).

"When concrete is subjected to a compressive fiber stress of 700 lb., the immediate fiber deformation is only about 50 per cent of what it will be if the stress is maintained for three weeks. Within 24 hours after the application of the load, the deformation has increased an additional 20 per cent; during the first hour the deformation may show a change of 5 per cent or more. Furthermore, the recovery of the deformations, after the removal of the load is slow and not complete.

"This fact is mentioned here to show the importance of taking all the deformation readings at approximately the same time after the load is applied, and in each case a new set of deformation readings under zero load must be taken just before each load application."

The Flow of Concrete under Sustained Loads, by E. B. Smith, Proc. Am. Conc. Inst., vol. 13, 1917.

In this paper, E. B. Smith of the U. S. Bureau of Public Roads, Washington, D. C., described time tests of compression cylinders and beams, using gravel and limestone as the large aggregates. The proportions in each case were 1:2:4, and in each group there were unstressed specimens which served as controls.

In the compression tests, the cylinders were 3 in. in diameter by 24 in. long and were loaded to a stress of 840 lb. per sq. in. Cylinders were cured in the open air and sprinkled once daily for the first week. Load was applied at the age of 28 days. Deformations were measured over a 20-in. gage length. The load was released after having been sustained for 50 days. In the following table are given values of the total deformations (elastic deformation plus flow):

It is significant that the deformation of the gravel concrete was 20 to 30 per cent greater than that of the limestone concrete.

The beams were 5 in. wide by 8 in. deep, effective depth 7 in., clear span 10 ft., steel ratio approximately 0.75 per cent, central load 830 lb., Load was sustained on the gravel beam for 34 days and on the limestone beam for 48 days, after which the load was completely removed. The concrete deformation in the gravel concrete beam was about 20 per cent greater than that for the

Days of Sustained Load	Total Deformation Due to Load, % of Length	
	Gravel Concrete	Limestone Concrete
0	0.042	0.032
3	0.087	0.060
14	0.126	0.095
50	0.157	0.124
50 Load released	0.119	0.090

limestone concrete beam, although the limestone concrete beam showed a slightly greater deflection. The concrete deformations had increased about 100 per cent within the given loading periods. Steel stress due to load increased from about 11,000 lb. per sq. in. to about 18,000 lb. per sq. in. in 34 days for the gravel concrete.

Following are some of the conclusions drawn by E. B. Smith:

The law of concrete flow is asymptotic. The flow continues at a gradually decreasing rate and yet is an appreciable amount during three to four weeks. It then continues more slowly for an indefinite period, but this change is small.

The total combined effects of shrinkage and flow in compression may amount to as much as 0.2 per cent.

The effect of flow within the material itself is either to relieve the stress condition, if the construction and loading make this possible, or to gradually change the length or position of the member.

The magnitude of flow deformations vary quite largely with the kind of aggregate and the mixture.[1] The modulus of elasticity[2] of concrete is different for each mixture and aggregate. It changes and decreases in value with time and as the flow deformations increase.

In the case of a reinforced concrete beam, the effect of flow in the concrete is to lower the position of the neutral axis, thus enlarging the compressive cross-section area and relieving the unit stress value. More stress is thrown on the steel.

In the discussion that followed F. R. McMillan brought forward data to show that in columns he had investigated in building in service, steel stresses as high as from 20,000 to 36,000 lb. per sq. in. were developed, which latter stress corresponds to a deformation of about 1½ in. per 100 ft.

Design of Reinforced Concrete Slabs, Trans. Am. Soc. C. E. Discussion by F. R. McMillan, vol. 80, 1916, p. 1743.

Two groups of test results not previously discussed in this paper are briefly reported. Tests of the first group were made on four 4 by 5-in. beams with a

[1]The great effect of moisture conditions was evidently not appreciated at that time.
[2]Here is meant the ratio of unit stress to total deformation, after any given period of time.

42-in. span. All concrete was of a 1:2:4 mix. Measurements were made on an 8-in. gage length spanning the center section of the beams. Pertinent data follow; shrinkage has been eliminated from the results by means of observations on similar unstressed beams.

TIME YIELD OF REINFORCED CONCRETE BEAMS, GROUP 1 (MCMILLAN)

Beam No.	Coarse Aggregate	Reinforcement	Calculated Stress, ($n = 15$) Lb. per Sq. In.	Age When Load Applied
11	pea size limestone	1⅜″ sq. twist	670	33 days
13	pea size limestone	2 5/16 pl. round	650	33 days
118	¼″ to ¾″ limestone	1⅜ pl. round	730	103 days
121	sandstone	1⅜ pl. round	730	103 days

Beam	Initial Unit Deformation, Inches	Relation of Total Deformation to Initial Deformation			
		30 Days	90 Days	200 Days	530 Days
11	0.000335	2.49	2.66	3.50	4.09
13	0.000302	2.48	2.78	3.28	3.74
118	0.000246	2.40	2.84	3.29	....
121	0.000180	1.57	2.08	2.68	....

In the second group were beams and slabs with dimensions and loading schedules shown below.

Specimen	Mix and Coarse Aggregate	Time Kept Wet, Days	Depth, In.	Width, In.	Effect. Depth, In.	Span In.	Steel Ratio	Age at Dead Load Application Days
A Beam	1:2:4 limestone pea size	1	5.00	4	4.50	42	0.0085	33
B Beam	1:2:4 limestone ¼″ to ¾″	not wet	5.50	30	4.62	144	0.0080	41
C Slab	1:2:4 limestone ¼″ to ¾″	31	4.87	120	4.35	120	0.0046	135
D Slab	1:2.50:4 limestone pea size	57	3.00	96	2.62	72	0.0049	59

Important live load variations:
Beam A. Dead load and 860 lb. at center applied at age of 33 days.
Beam B. Loaded at ⅓ points, 500 lb. at 80 days, 1500 lb. at 88 days.
Slab C. 50 lb. per sq. ft. at 174 days after dead load application.
100 lb. per sq. ft. at 275 days after dead load application.
Slab D. 50 lb. per sq. ft. at 68 days.
100 lb. per sq. ft. at 71 days.
Live load removed at 82 days.
Live load reapplied at 94 days.

The results of tests on beams and slabs of the second group are shown in the following table.

TIME YIELD OF BEAMS AND SLABS, GROUP 2 (MCMILLAN)

Type Loading	Unit Instantaneous Deformation			
	Specimen A	Specimen B	Specimen C	Specimen D
Dead load	....	0.000200	0.000076	....
Live load				
0 days	0.000280	....	....	0.000042
3 days	....	....	....	0.000065
46 days	....	0.000035	....	....
54 days	....	0.000075	....	....
174 days	....	....	0.000058	.....
275 days	....	....	0.000043	....

Type Loading	Ratio, Total to Sum, Instantaneous Deformations			
	Specimen A	Specimen B	Specimen C	Specimen D
Dead load				
50 days	2.57	....	2.90	2.24
100 days	2.68	2.32	....	2.66
200 days	3.39	3.11	3.20	3.08
650 days	4.25	3.63	4.00	4.11
950 days	4.60	3.71	....	4.39

Consulting the results of the two groups of tests it is seen that the total deformation after a six months of sustained load is roughly three times the initial deformation, and after two years is roughly four times the initial deformation. From another point of view, the ratio of moduli for steel and concrete, n, which ranged from 9 to 15 at the time of initial loading, may increase to 20 or 30 in a few months, and from 35 to 60 in 2 years.

It is noteworthy that here for the first time were reported tests extending over a period of two years or more, and the discovery was made that flow was still taking place at the end of these extended periods of observation.

Extensometer Measurements in a Reinforced Concrete Building Over a Period of One Year, by A. R. Lord, Proc. Am. Conc. Inst., vol. 13, 1917, p. 45.

In this paper A. R. Lord reports the results of tests made in 1914 and 1915 on a 25 by 26-ft. flat slab in a reinforced concrete warehouse located in Chicago. The load, which was twice the design load, was left in place for about a year, and strain-gage measurements were taken periodically during this time. The slab was 10 in. thick and the concrete was a 1:2:4 mix. What were believed to be corrections were made in the results for temperature changes (based probably on air temperature and the coefficient for steel) but the deformation reported include shrinkage. The test load was a uniformly distributed layer of sand.

The results are best stated in the form of general conclusions, as follows: The ratio of deformations at 1 and 55 days is about 1:2.5 and the ratio of de-

formations at 1 and 230 days is about 1:5 (results not dissimilar to those of previous tests).

The continued deformation of the concrete was noted, but the rate of deformation of the steel was not so great as that of the concrete, showing that although flow took place continuously, shrinkage operated to reduce the tensile stress observed in the steel (see McMillan's paper on Time Tests of Concrete).

Due to flow, and therefore increased deflection in regions of negative moment, a shifting of the moment took place, resulting in a still greater increase in deflection at regions of positive moment.

The increase in total moment figured from steel stress was relatively small.

The "straight line theory" of reinforced concrete is inapplicable to the conditions surrounding this test even with the low stresses present in the concrete and steel.

Plasticity and Temperature Deformations. by S. C. Hollister, Proc. Am. Conc. Inst., vol. 15, 1919, p. 127.

Load deformation data are presented for tests made at Purdue University on a narrow reinforced concrete slab. Under the conditions of the test, which are not reported, the elastic deformation is about the same as the "plastic" deformation for all magnitudes of load, for both steel and concrete. Plastic deformations evidently include shrinkage. Plasticity is defined as that deformation which does not disappear upon the removal of load, i. e., permanent set.

A Study of Column Test Data, by F. R. McMillan, Proc. Am. Conc. Inst., vol. 17, 1921, p. 150, and Discussion by M. B. Lagaard, loc. cit. p. 172.

Incidental to a general discussion of reinforced concrete spiral-wound columns, McMillan reviews briefly his own and other flow data.

In his discussion of McMillan's paper, Lagaard presents more recent data regarding the column flow tests on a building at the University of Minnesota (which tests have been previously mentioned in this paper in connection with McMillan's discussion of other papers). Also there are given the results of tests on three columns in reinforced concrete warehouses.

The columns in the building at the University of Minnesota were still decreasing in length (due to flow and shrinkage) after a period of six years. The tests on these columns were begun after they had been in existence for about two months, so that observed deformations do not include those due to shrinkage and to flow for that preliminary period. Assuming a constant modulus in the steel of 30,000,000 lb. per sq. in., the increase in stress for the six years varied among the several columns from 36,000 to 45,000 lb. per sq. in. The percentage of vertical steel in these columns varied from 0.7 to 2.1.

In one warehouse, in which the columns had 3.7 per cent of vertical steel and 1.6 per cent spiral hooping and the tests were begun at an age of three weeks, the increase in vertical steel stresses for an observed period of 4½ years was 14,000 to 15,000 lb. per sq. in. It is also interesting to note that the lateral shrinkage during this period corresponded to steel stresses of from 11,000 to 15,000 lb. per sq. in. Another noteworthy feature of these tests is that in one column the vertical gage lines in the concrete showed about 20 per cent more

deformation than did those in the steel. This may have been due to the fact that the concrete and steel gage lines were not in the same vertical element and some bending was taking place.

Tests on another warehouse column containing 4.5 per cent vertical steel and 1.0 per cent spiral, showed, after nearly 5 years, a concrete deformation of 0.046 per cent and a steel deformation of 0.029 per cent, the latter corresponding to a stress of about 9,000 lb. per sq. in. In this column the vertical elements in which the concrete gage lines were located made angles of about 60 deg. and 120 deg. with the element containing the steel gage lines.

Conclusions stated by Lagaard are: High compressive stresses are likely to exist in the vertical steel reinforcement of concrete columns due to the combined effects of load, shrinkage, and time-yield.

The transverse steel will also have high compressive stress due to the action of shrinkage.

The compressive stresses in the vertical steel will be lessened when the percentage is increased.

Fatigue of Concrete, Discussion by H. F. Clemmer, Proc. A. S. T. M., vol. 23, 1923, p. 339.

In connection with what he calls "static load fatigue," Clemmer reports the following of cantilever beams subjected to static loads which produced stresses of various percentages of their moduli of rupture. "The critical percentage of stress which caused failure was found to be 71 per cent. A number of beams failed at the end of approximately 105 hours under a load causing a stress of 71 per cent of the breaking stress. With a stress of 84 per cent, the beams failed at the end of 14 hours." Clemmer also states that most of the set due to a static load is temporary and is recovered after a time.

Plastic Yield, Shrinkage and Other Problems of Concrete and Their Effect on Design, by O. Faber, Min. of Proc. Inst. C. E. (Great Britain), vol. 225, 1927-28, Part I.

Faber reports the results of laboratory tests made in 1926-27 on six simply supported beams, each 2 in. wide, 5 in. deep, effective depth 4½ in., of span 15 ft. The concrete was approximately a 1:1¾:3½ mix with a water cement ratio of 0.69. The tension steel consisted of two $\frac{3}{16}$-in. round rods in all of the beams, with the addition of a like amount of compression steel in beams 5 and 6. As tested the dead load of each between supports was 150 lb. At 28 days test cubes of the concrete of beams 1 to 4 exhibited a compressive strength of 3400 lb. per sq. in., test cubes for beams 5 and 6 gave 3300 lb. per sq. in. and test cylinders for beams 5 and 6 gave 2600 lb. per sq. in. The steel for beams 1 to 4 had a yield point of 50,300 lb. per sq. in. and that of beams 5 and 6, 28,600 lb. per sq. in. Storage conditions as regards temperature and humidity are not stated.

All beams were stored in air continuously, except No. 4 which was kept wet after the age of 30 weeks. Beams 1 to 4 were placed upon their end supports and loaded at age of 4 weeks; beams 5 and 6, at the age of 5 weeks. Beam 3 was unloaded at the age of 30 weeks. Deflection measurements were made periodically beginning with the time of loading. Deformations in steel and

concrete were observed on beams 3 and 4 between the ages of 10 and 18 weeks and on beam 6 between the ages of 6 and 15 weeks.

The following table gives the centrally applied live loads for the several beams, and also gives calculated values of concrete and steel stress, as well as calculated and observed deflections. The calculated stresses and deflections are based upon an assumed straight line variation of compressive stress in concrete with no tension in concrete, and upon modulus of elasticity of 4,000,000 lb. per sq. in., $n = 7.5$. The observed deflections are those that took place at the instant the loading tests were begun. It will be noted that the observed instantaneous deflections are roughly 0.6 of the corresponding calculated values.

BEAM LOADINGS, STRESSES AND DEFLECTIONS (FABER)

	Beam No.					
	1	2	3	4	5	6
Live load, lb.	0	30	60	90	0	30
Calculated Stresses, lb. per sq. in.						
Concrete	676	946	1216	1487	608	851
Tension steel	13750	19250	24750	30250	13640	19100
Comp. steel	....	....	....	....	2540	3550
Calculated Instant. Deflection, in.	0.474	0.622	0.770	0.918	0.47	0.61
Observed Instant. Deflection, in.	0.30	0.35	0.40	0.50	0.29	0.37

The behavior of the beams under sustained load is indicated by the two following tables, taken from graphs prepared by Faber, the first giving observed deflections and the second giving deformations of top and bottom fibers at various ages. The values show the combined effect of plastic yield and shrinkage. Ages are counted from the day the specimens were cast. Consulting the tables, it is seen that both deflections and compressive deformations (top) increase substantially as time goes on. Further, for the tensile fibers (bottom) of some of the beams, the effect of shrinkage nearly offsets that of plastic yield.

BEAMS UNDER SUSTAINED LOADS (FABER)

Observed Deflections, Inches

Age, Weeks	Beam Number					
	1	2	3	4	5	6
4	0.30	0.35	0.40	0.50	....	....
5	0.34	0.41	0.54	0.65	0.29	0.37
10	0.57	0.67	0.89	1.01	0.55	0.67
20	0.78	0.89	1.16	1.25	0.69	0.80
25	0.80	0.93	1.19	1.26	0.71	0.82
30	0.81	0.98	1.22	1.31	....	....
40	0.85	1.01	....	1.40	....	....

BEAMS UNDER SUSTAINED LOADS (FABER)

Observed Deformations, Top and Bottom Fibers
(Positive Signs Indicate Elongations)

Age, Weeks	Observed Deformations, Per Cent of Gage Length					
	Beam 2		Beam 3		Beam 6	
	Top	Bottom	Top	Bottom	Top	Bottom
6	....	....	....	....	0	0
8	....	....	....	....	—0.0081	+0.0050
10	0	0	0	0	—0.0133	+0.0062
12	—0.0125	+0.0005	—0.0130	+0.0026	—0.0187	+0.0075
14	—0.0145	+0.0035	—0.0168	+0.0053	—0.0240	+0.0100
15	—0.0150	+0.0052	—0.0171	+0.0070	—0.0250	+0.0105
18	—0.0220	+0.0040	—0.0240	+0.0074	....	....

Assuming the deflections to vary directly with the deformations, Faber has calculated the probable deformations of the top fibers of four of the beams up to the age of 32 weeks, due to combined shrinkage and plastic yield as well as the corresponding strains which took place when load was applied at the age of four weeks. These values are compared in the following table. It is instructive to observe that the calculated deformation at the end of 32 weeks due to plastic yield plus shrinkage is on the average nearly three times the calculated deformation taking place at the time the load was applied.

COMPUTED TOP FIBER DEFORMATIONS IN BEAMS

Between Ages 4 and 32 Weeks (Faber)

Beam No.	Calculated Elastic Strain, In. Per In.	Top Deformation Due to Shrinkage and Plastic Yield, In. Per In.
1	0.00017	0.00052
2	0.00024	0.00066
3	0.00030	0.00084
4	0.00037	0.00087

From a study of these tests and other data Faber separates the element of shrinkage from plaster yield and draws the following conclusions:

1. At a stress of 1000 lb. per sq. in., the plastic yield in 36 weeks is approximately equal to the total shrinkage and each is approximately twice the elastic strain.

2. The plastic yield varies with the elastic strain, being roughly double its value, while the shrinkage remains constant.

3. The total deformation in 36 weeks is roughly 6 times the elastic strain at 600 lb. per sq. in., about 5 times that at 1000 lb. per sq. in., and about 3½ times that at 1400 lb. per sq. in.

4. In beam No. 6, with compressive reinforcement, for the period 5 to 22 weeks, the calculated increase in stress due to shrinkage and plastic flow was 11,700 lb. per sq. in. for the compression steel, and 3,800 lb. per sq. in. for the tension steel. For beams 2 and 3, the calculated additional stresses in the steel from 4 to 36 weeks were 1,400 and 4,300 lb. per sq. in. respectively.

Conclusions

A review of the abstracts of the preceding pages giving the results of tests by various investigators justifies the following general conclusions:

1. All concrete under the action of sustained stress flows or plastically deforms for a long period of time, certainly for several years. This is true regardless of whether the concrete be plain or reinforced and is true for beams and slabs subjected to bending moment as well as to columns subjected to direct compression.

2. Regardless of the character of the concrete or of the conditions surrounding it, flow increases with the magnitude of the sustained stress, as well as with time. Other things being equal the flow is proportional to the instantaneous deformation or that which takes place as the load is applied.

3. When concrete is in air, shrinkage continues for a long period of time, the amount of shrinkage depending upon the dryness of the atmosphere, but being independent of stress conditions. This is true for reinforced as well as for plain concrete.

4. In a reinforced concrete column the combined effect of shrinkage and flow under sustained load is gradually to relieve the compressive stress in the concrete until the yield point of the steel is reached. There is evidence that under normal load conditions the spiral hooping of a column surrounded by dry air, due to lateral shrinkage, is always in compression.

5. In beams and slabs reinforced in the normal manner the combined effect of shrinkage and flow is gradually to lower the neutral axis at sections of positive moment and to raise the neutral axis at sections of negative moment. This results in an increase in deflection, in a reduction of compressive stress upon the extreme fiber of the concrete, and in an increase in the tensile stress in the steel. In continuous and restrained beams and slabs there is also brought about, due to flow, a gradual redistribution of moments tending to equalize those of opposite sign.

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Basis of Design For Hurricane Exposure

Report of Committee 308

BY ALBERT SMITH, AUTHOR CHAIRMAN*

This brief report prepared by Mr. Smith has been submitted to the critic members of his committee: D. C. Coyle, Hardy Cross, W. J. Knight, G. A. Maney, L. E. Ritter and W. J. Spiker and since redrafted. The report is here supplemented by the Author-chairman's discussion and discussion invited for subsequent publication, from the critic members of the committee and others.—Editor.

The structural frame shall be designed to resist all the force of the wind on the total exposed surface.

In computing wind loads, the following unit pressures, varying with the height above sidewalk shall be used:

0 ft. to 200 ft.—20 lb.
200 ft. to 300 ft.—25 lb.
300 ft. to 400 ft.—30 lb.
400 ft. to 500 ft.—35 lb.
500 ft. —40 lb.

except that structures located completely within a solidly built up section may be designed for 10 lb. of pressure below the highest complete shelter found within 400 ft. of its walls.

Spread footings, pile footings and caps and caissons shall be so designed that the loads and stresses from the combination of dead, live, and wind loads shall not exceed by more than 50 per cent the design units for dead and live alone.

All columns whose vertical wind load does not exceed 20 per cent of the dead and live load may deliver what wind moment they carry in their lowest section into square spread footings,

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square pile clusters and to caissons without increasing such footings for wind. Where the footings are rectangular they shall be examined for soil pressure, pile load and steel and concrete stresses and if the increase from wind vertical and moment exceeds 50 per cent of the dead and live units they shall be redesigned.

All members of the structural frame whose connections cause them to bend when the building deflects under wind may be regarded as sharing in the resistance to the wind.

The structural frame shall be regarded as a unit in resisting the wind pressure by the bending of its members and the moments in all joints of the structure shall be assigned in proportion to the stiffness of the joint which depends on the stiffness of the members connected at such joints. Where two members share the moment at a given joint, the moment shall be divided between them in proportion to their respective stiffness.

The moments so found in the members may be reduced to the face of the column, for beams, and to the top and bottom of the beams, for columns. The connections of all members shall be designed for these reduced moments by the stress units hereinafter specified.

The stiffness of the joints of braced bents at floors where the bracing begins or is stopped, is less than that of joints at the same floor of continuously braced bents whose members have the same properties. At offsets and at all places where shallow or no beams are placed between columns which have stiff girders in floors above, and where downward going bracing bays start, an uncertainty exists as to the amount and location of the horizontal wind shear which will be transmitted.

This horizontal wind shear shall be assumed to be transmitted at three floors and in assigning stresses to girders, columns, and horizontal floor bracing, an excess of at least 75 per cent of the detoured wind shear must be divided among these three floors to cover this uncertainty.

Where the bracing for a tower apparently could be stopped at the main roof line and transferred to other lines the tower bracing shall be carried at least two stories below the top of the bracing of the enlarged section below.

Where a column of a braced bent is carried on a girder, allowance shall be made in distributing the stresses, for the deflection of the girder, as affecting both the moment and the vertical load of the riding column and the columns adjacent in the bent.

If columns designed for dead and live loads are found to be inadequate for combined stresses of dead, live and wind loads, they shall be redesigned to make them adequate by increasing the size, or adding to the vertical steel or by increasing the spiral or by adding vertical steel wired to the spiral at the corners of the column or by a combination of these devices.

Where the spandrel beams are relatively shallow, the wind stresses in interior flush beams and joists will be relatively large. Such flush beams and joists shall be redesigned for the combined stresses from dead, live and wind loads.

DESIGN UNITS

Beams—Where no wind rods are used, the capacity of beams to resist wind moment shall be computed for a fibre stress of 500 lb. per sq. in. in the bottom fibres of the beam, neglecting the steel provided for static loads. Where the moments exceed the capacity of the concrete, steel wind rods shall be added of such cross sectional area that, taking the distance between the centers of the top and bottom rods as the effective depth the fibre stress shall not exceed 27,000 lb. per sq. in. for medium grade steel, neglecting the resistance of the concrete.

The bond of these wind rods shall not exceed 150 per cent of the bond unit for static loads, except that where a length curved on a radius of not less than 10 in. is inserted in the column the bond of such curved length may be 225 per cent of the bond unit for static loads. Where straight wind rods project through the column into the beams, the bond in the column must be equal to the sum of the compression in the steel on one side and the tension on the other. Wind rods shall project into the beam to the point at which the combined positive static and positive wind moment

will not stress the positive static steel more than 150 per cent of the static steel unit stress.

For the combined static and wind shear the unit of diagonal tension and stirrup stress shall be taken as 150 per cent of the units for static load alone. Girders, whose ends are connected to columns by wind rods, shall be counted as fulfilling the condition of "Special Anchorage" for static load.

Columns—The combined stress, consisting of dead and live load, vertical wind load, and the wind moment transformed into equivalent centric load, shall not exceed 150 per cent of the capacity by code for dead and live loads only. The wind moment in the column at the top or bottom of the beam may be transformed by the formula Equiv. Cent. Load $P_{WM} = \frac{AMv'}{I}$, where A is the area inside the spiral, v' the distance of the spiral from the center of gravity and I the moment of inertia of the total section of the column.

DISCUSSION BY ALBERT SMITH, AUTHOR-CHAIRMAN

WIND PRESSURE UNITS

To ESTIMATE the expectable velocity of wind which may produce the maximum pressure on a building, two rather indefinite factors must be applied to the records of maximum velocity by three-cup anemometers or four-cup anemometers corrected to three-cup equivalents—one for height of instrument, and one for gust intensity. See Table 1.

The reported velocities are average velocities for five minutes. In that time the record of pressures shows that the velocities have varied from 25 per cent of the average to 175 per cent of the average. On the face of it, we should add 50 per cent to the corrected reports of average readings to determine the proper design pressure. These gusts, however, may be of small cross section.

It is probably sufficiently proved that near the ground the width of gusts is small. It is not proved that at a height of say 150 ft. in a clear air way the width of gusts is small. Fleming[1] quotes Stanton's conclusions from observations taken on the high level footway over the draw-span of the Tower Bridge at

[1]Wind Stresses in Buildings, p. 53.

TABLE 1—MAXIMUM REPORTED WIND VELOCITIES, DEPARTMENT OF AGRICULTURE—BULLETIN W-1926, VOL. 1, 2, 3.

Years During	City	4 Cup	Corr. 3 Cup	50 Per Cent Incr.	Elev. Station
47	Albany	70	57	86	115
48	Augusta, Ga.	62	50	75	77
50	Boston	72	58	87	188
32	Cairo	84	67	101	93
50	Charleston	106	85	128	92
42	Charlotte, N. C.	72	58	87	62
49	Cheyenne	78	63	95	101
48	Chicago	84	67	101	131
52	Cincinnati	54	44	66	51
51	Cleveland	73	59	89	201
44	Columbus	78	63	95	222
7	Dallas	62	51	77	227
49	Davenport	72	58	87	79
10	Dayton	62	51	77	173
48	Denver	75	60	90	113
51	Detroit	87	70	105	258
49	Duluth	78	63	95	47
47	Eastport	78	63	95	85
49	Erie	71	57	86	166
25	Evansville	77	62	93	116
41	Fort Smith	74	60	90	94
22	Ft. Worth	66	54	81	114
	Grand Rapids	66	54	81	87
42	Helena, Mont.	70	57	86	112
51	Indianapolis	71	57	86	230
49	Louisville	74	60	90	234
48	Memphis	75	60	90	97
25	Minneapolis	84	67	101	208
46	New Orleans	86	69	104	84
37	New York	96	77	115	454
30	Philadelphia	75	60	90	182–367
49	Pittsburgh	70	57	86	410
49	Portland, Me.	61	50	75	106
51	Portland	55	45	68	106
16	Providence	82	66	99	251
34	Pueblo	64	52	78	86
48	St. Louis	80	64	96	303
32	St. Paul	102	81	122	261
52	San Francisco	64	52	78	243
44	Sandusky	69	56	84	67
50	Savannah	88	71	107	194
29	Seattle	64	52	78	250
19	Syracuse	66	54	81	113–79
49	Toledo	84	67	101	...
41	Vicksburg	62	51	77	73
36	Walla Walla	65	53	80	65

London. Stanton declared that he found that gusts might occur whose intensity would be practically uniform over the whole structure. Where a comparatively steady high velocity wind is

blowing between 1000 ft. and 2000 ft. above the earth, the gusts of which we have record originate by the deflection of masses of air from the swiftly moving stream toward the earth. It is not reasonable to suppose that these deflected masses are splinters of air current less than 100 ft. wide. On the contrary, it is logical to assume that these are very much larger masses, which as they approach the earth are split up successively by the roughness of the surface of approach and the infiltration of the slower currents below. The maximum velocity at the center of the gust is probably indicative of the velocity of the main air current above. Thus a one minute velocity in New York of 91 mi. per hr. at 454 ft. elevation shows a 30 second velocity much greater, and is ample justification for assuming that the main current of air is traveling much faster than 100 mi. per hr.

Pressure is determined by applying a coefficient to the square of the velocity. Dryden and Hill found at the Bureau of Standards[2] that a tower model whose proportions were 1:1:3 received a total pressure which can be expressed as $P = .00384\ V^2$, and they predict that higher towers will show a larger coefficient. Other observers have found that long plates receive a higher unit force than square plates.

It is my contention that we should start with 40 lb. (100^2 x .004) for say 1500 ft., and not diminish it until we reach the elevation at which we are sure that the width of the gust having an average pressure of 40 lb. is less than the width of the structures to which we propose to apply the unit. I believe that this point cannot safely be taken as higher than the 500 ft. point above sidewalk, or 300 ft. above the base of the clear air-way

I propose that the pressure units should be 40 lb. above 500 ft. from sidewalk, 35 lb. between 400 ft. and 500 ft., 30 lb. between 300 ft. and 400 ft., 25 lb. between 200 ft. and 300 ft., 20 lb. between 100 ft. and 200 ft., and 15 lb. below 100 ft.

I have no confidence that there is any place in the country where these pressure units will not be exceeded during the next 20 years, but I am confident that in buildings carefully designed for the stress units set forth by the committee and for these pressures, there will be no overstrain that will damage the structural frame.

[2]Scientific Paper No. 523, p. 731.

COMPUTATION OF WIND STRESSES IN BUILDINGS

In a building of 22 stories, having 45 columns, the wind causes stresses whose total number for two directions of wind is nearly 10,000. A large part of these stresses are not important to the designer. A few upper stories may be dismissed from consideration for all stresses. Most intermediate columns have minor axial stresses from wind which need be computed only for one or two stories. With all these deductions the number of stresses to be computed and marked on a set of framing diagrams is very large. Any method which will give us, reliably, the wind stresses in a building which will not be exceeded by 5 per cent nor be in excess more than 10 per cent should be quite satisfactory to the designer.

Several short methods of calculation for single bents have been proposed. Some appear likely to have a maximum error of only 5 per cent. If however, the amount of wind which will come in this bent is not determined within 25 or 30 per cent the accuracy of distribution within the bent is of little use.[3]

Any structure must be regarded as a whole in computing its wind stresses, and when the building is unsymmetrical in any respect, and is of great height, the labor of making the closest practicable approximation to the true stresses is not only justified, it is essential.

For a symmetrical building with all bays in the same direction equal, and having intermediate columns so turned that their stiffnesses are proportionate in the bents in which they are intermediate, a fair approximation of the wind moments at the joints may be made by the following assumptions.

a. All points of contraflexure at the middle of the lengths of all columns and beams.

b. Equal vertical wind shears in all beams.

Each floor of a building deflects horizontally as a unit under wind pressure. If the building is symmetrical, both for layout

[3]Mr. Fleming in his very interesting book, "Wind Stresses in Buildings," (John Wiley, 1930) quotes Prof. Burr's statement that "So long as the stresses found by one legitimate method of analysis are provided for, the stability of the structure is assured." We should not allow this truth, so excellently stated, to make us satisfied with crude methods of finding wind stresses in buildings. After all, are they "legitimate?" I take it Prof. Burr did not mean that if the stresses as arbitrarily divided between 40 or more paths are provided for, the structure is safe. Nor if a weight be hung on a thread and a rubber band of the same length, that the factor of safety is the weight divided by the sum of the two ultimate strengths. The designer is charged with the duty not only of supplying enough strength, but also of putting it in the right place.

and framing, all points along the wind exposure of the floor deflect equally.

Even in the most symmetrical building, however, computations made with the assumptions given above must have some approximate corrections. All buildings have, in one or the other direction of the wind, much shallow framing whose resistance to the wind, if it is symmetrical, may be subtracted directly from the total moment at the given floor.

Even in the most symmetrical buildings the I of the end columns of the principal bents may be so much less or so much more than one half that of the intermediate columns of these bents, that the calculations by the method outlined above will be seriously in error.

Either the columns should be adjusted for stiffness or an allowance should be so made in the stresses that the designer can be confident that his assumed stresses will not be exceeded.

Where a building is unsymmetrical but has equal column spacing, the eccentricity of the wind load with respect to the stiffness of the building can be computed and adequate allowance made in the stresses without too greatly loading the design.

When, however, for a given direction of wind, the beam spans vary widely or the wind resistance is very unsymmetrical or the beam and column stiffnesses are made disproportionate to their wind function by other load requirements, or as often happens, all of these unfavorable conditions exist, we must use some more powerful method of computing the distribution of the wind stress among all the resisting members with a reasonable degree of accuracy.

The method shown herewith has been developed in tabular form so that engineering draftsmen might fill in the tables and compute the stresses and learn to understand the method while they computed. For a building with 45 columns we estimate that the tabulation of $\frac{1}{R}$, the true K, and the distributed moments for one floor can be made by one man in two and one half days. The total work for three floors, in two directions and the placing of computed and interpolated stresses on the general stress diagram is of the order of two men for two weeks.

There will still remain the examination of columns for wind stress, and in some cases the correction of many of them, and the design of the brackets for a steel frame, or the wind rods for a concrete building. A very competent designer, with a draftsman helper, would probably find plenty to do for a week after all stresses are known.

The wind stresses in Eddystone Belmont were first laid out by the approximate method, roughly corrected for eccentricity of column spacing and column and beam stiffness. These figures were used in the design for estimate, and contract. As always, after the estimate plans go out there was plenty of time for computing more exactly. We found, by calculation, that in the major stresses our allowances were ample, but that we had in some bents used an excess of steel, and carried the wind reinforcing too high, and in some bents we found our wind reinforcing deficient. A careful approximation of the stresses was more rapid than the calculation, but even though our allowance for error was large, we needed the reassurance of the calculation. For this building the calculations here shown might have been greatly shortened. The calculations for joists and for flush beams might have been lumped together in an approximate total without danger of a large error in the final moments in the important beams. These calculations after having been originally made with some abbreviations were expanded to a complete table to enable readers to study the relation between moments of joints with varying conditions.

AN EXAMPLE OF WIND STRESS COMPUTATION

To apply, with reasonable amounts of labor, the equation shown in Fig. 1, it has been found necessary to develop charts from which the factors $\frac{1}{I_c}$ and $\frac{I_B}{L_B}$ can readily be obtained.

I have made, but not given in this paper, the tabulation of the calculation of the I's of square columns varying from 11 in. to 33 in. in gross dimension and with spirals varying from 8 in. to 30 in. For these calculations the E for 3000-lb. concrete was taken as 3,600,000 and the n as 8 and for 2000-lb. concrete, as 3,000,000 and 9.6. The values of I are computed as:

$$I_{2000} = .0833D^4 + .008p\ (D - 3)^4$$
$$I_{3000} = .10D^4 + .008p\ (D - 3)^4$$

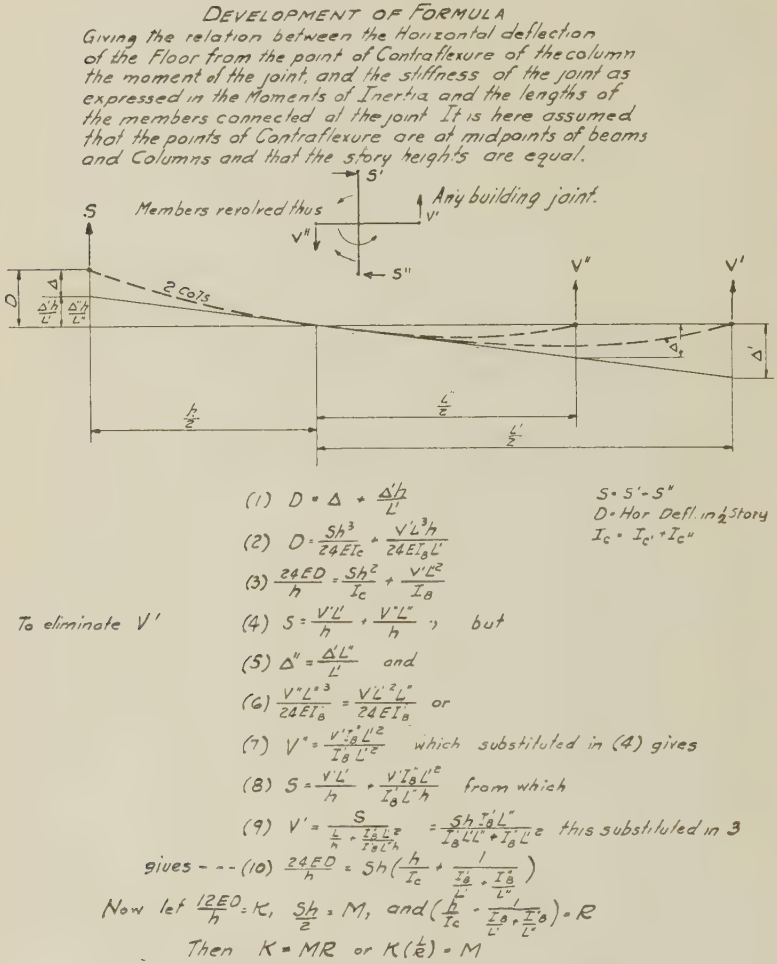


FIG. 1—DEVELOPMENT OF FORMULA

It is obvious that for the smaller values of p, the diameter of the steel ring is larger and the I of the steel proportionately greater than for the larger values of p, also that for large spirals the I of steel is greater than for the same percentages in smaller columns. The $.008p$ used here is a reasonable mean of these variations for the larger percentages. For the smaller percentages, the steel contribution to the I is small. In each square on this table, in addition

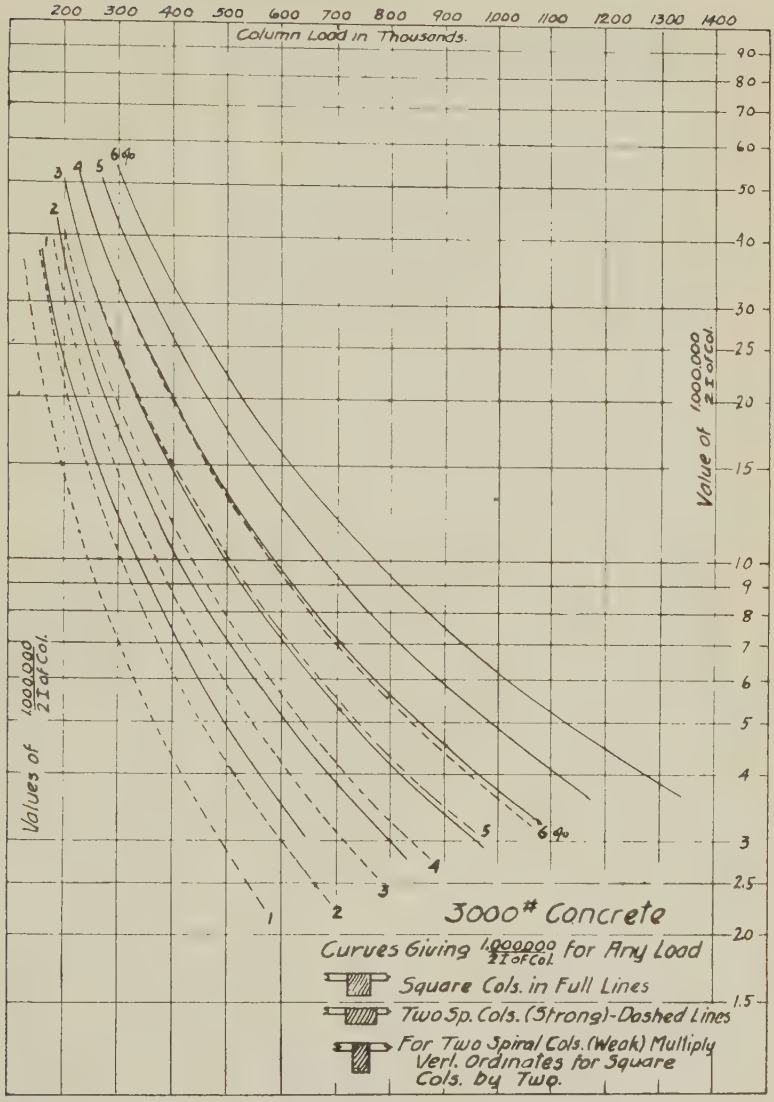

200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400
Column Load in Thousands.
90
80
70
60
50
40
30
25
20
15
10
9
8
7
6
5
4
3
2.5
2
1.5
Value of 1,000,000 / Σ I of Col.
Values of 1,000,000 / Σ I of Col.
3
4
5
6 & 0
2
1
3000# Concrete
Curves Giving 1,000,000 / Σ I of Col. for Any Load
Square Cols. in Full Lines
Two Sp. Cols. (Strong)-Dashed Lines
For Two Spiral Cols. (Weak) Multiply Verl. Ordinates for Square Cols. by Two.

FIG. 2

to the I in thousands, there are shown the maximum column load in thousands and 1000 times the reciprocal of $2I$. In Fig. 2 are shown the curves plotting 1,000,000 times these reciprocals against the maximum safe column load. Numbers obtained from these curves are to be multiplied by the story height in feet.

The use of the load to determine the reciprocal of the stiffness factor of columns leads to error because the curves are laid out for the minimum section which can be used to carry that load. If an excess of steel is used in any column, the $\frac{1{,}000{,}000}{2I}$ as determined by the actual load and the excessive percentage will be too large, and the stiffness factor will be too small. The effect of this error on the final moments is small but some effort has been made in using these curves to make the figures for $\frac{1{,}000{,}000}{2I}$ relative by roughly adjusting them to each other after the columns on Tables 2 and 3 have been filled in. The joints, whose final stiffness factors are most affected by these errors, are those whose moment is small and the most important stresses are very slightly affected.

Fig. 3 gives the beam curves from which $\frac{I}{1000L}$ can be obtained for spans ranging from 8 ft. to 40 ft., and for depths varying 9 in. to 48 in. The scale on the left edge permits the direct determination of the reciprocal for joints having only one beam. The curves are made for beams 8 in. wide and having only one flange. The I of the stem—without steel—is counted as $\frac{8d^3}{12}$, and the I of the one flange, with the steel, is counted as also equal to $\frac{8d^3}{12}$. For wider beams I have added $\frac{d^3}{12}$ to the I for each inch of increased width of stem, and for beams with two flanges I have added $\frac{8d^3}{12}$.

A table of coefficients is given for increase of stem widths for one and two flange beams.

It is, of course, quite impossible to make a rational determination of the I of concrete beams for stiffness. It is quite clear that the method here used is only approximate. Short spandrel beams, for instance, are likely to have very light reinforcement for dead and live loads. The final error from this cause is much reduced by the fact that the short beams have more wind moment than the long beams, and will certainly have as much wind reinforcing. Being short, their wind reinforcement will cover so large a portion of their length that the final error in the I is not so large as at first appears.

These assumed values for the I of beams tend to give us excessive stiffness for short beams. The practice here followed of using c. to c. of columns minus one quarter of the width of each column as the span of the beam tends to error in the other direction for short beams. The resultant of these errors is

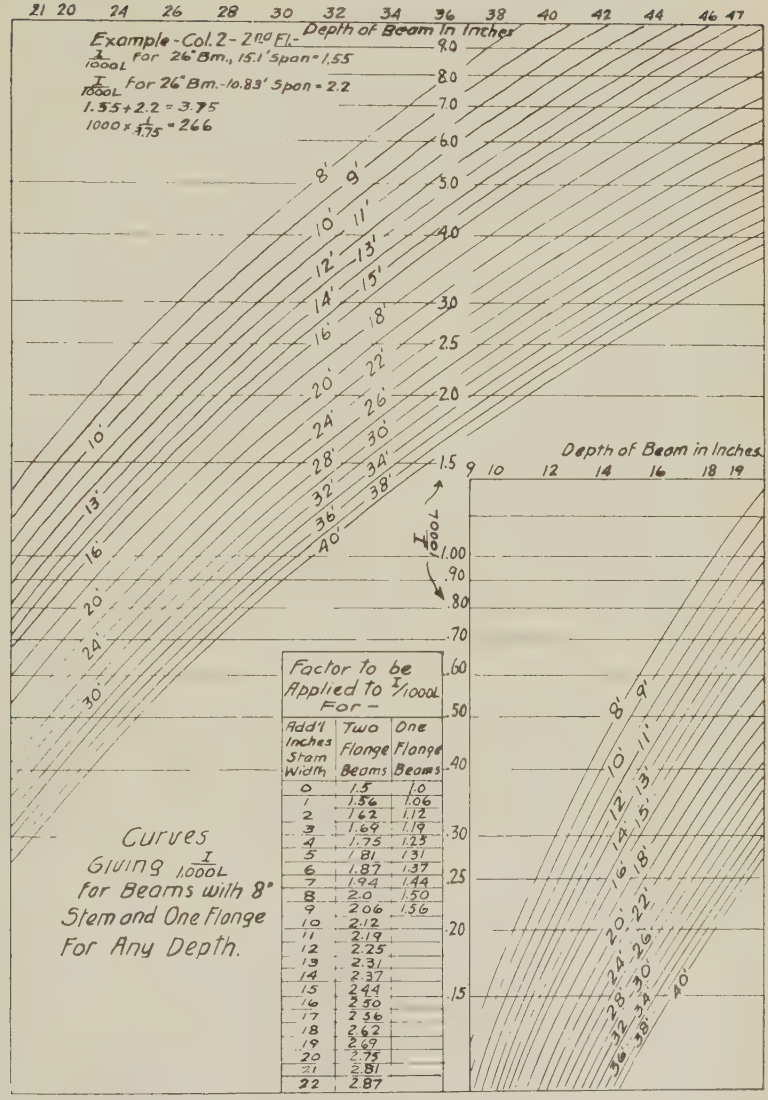

Example - Col. 2 - 2nd Fl. -
Depth of Beam in Inches
1/1000L for 26" Bm., 15.1' Span = 1.55
I/1000L for 26" Bm. - 10.83' Span = 2.2
1.55 + 2.2 = 3.75
1000 × 1/3.75 = 266
Depth of Beam in Inches
I/1000L
Factor to be Applied to I/1000L For —
Add'l Inches Stem Width | Two Flange Beams | One Flange Beams
0 | 1.5 | 1.0
1 | 1.56 | 1.06
2 | 1.62 | 1.12
3 | 1.69 | 1.19
4 | 1.75 | 1.25
5 | 1.81 | 1.31
6 | 1.87 | 1.37
7 | 1.94 | 1.44
8 | 2.0 | 1.50
9 | 2.06 | 1.56
10 | 2.12
11 | 2.19
12 | 2.25
13 | 2.31
14 | 2.37
15 | 2.44
16 | 2.50
17 | 2.56
18 | 2.62
19 | 2.69
20 | 2.75
21 | 2.81
22 | 2.87
Curves Giving I/1000L for Beams with 8" Stem and One Flange For Any Depth.

FIG. 3

probably quite small. The most serious question is whether the difference between $\frac{16d^3}{12}$ and $\frac{24d^3}{12}$ truly represents the difference between the stiffness of spandrel and interior beams. After much computation the I's here used seem to the writer to give the best relative results for varying depths, widths, and flanges.

Tables 2 and 3 show the calculation work tabulated for Eddystone Homes, a 22-story reinforced concrete building in Chicago. See Fig. 4. The wind action on this building is so intricate that we regard it as a fair sample of the worst case. It has been worked out here for the second, the tenth and the seventeenth floors. The columns change size at the ninth and at the sixteenth floors, and since most of the columns are rectangular and the beam span change is unusually large, the stiffness factors and the moments are not proportionate to those at the second floor. Probably in most buildings with square columns and uniform depth of spandrels, two, instead of three calculations would be sufficient, and the second one might be made only partially. The complete tabulation and calculation of these three floors give us a very interesting study of the distribution at any one floor, and the change of distribution caused by the framing changes in higher stories.

In Tables 2 and 3 and the headings explain themselves as terms in the formula of Fig. 1. The multipliers 1000 and 1,000,000 are used for convenience in tabulating.

When the column $\frac{1}{R}$ is filled out we can solve the equation $K = M/\frac{1}{R}$ to get the average value of K. In a symmetrical building the average K would be the true K; that is the deflection of the floor would be the same at all points along the line of exposure. If, however, the building is stiffer at one end than at the other, there would be less deflection at that end, and K, which is D times a constant, will vary from end to end. We may regard the stiffness units as similar to fibres in a beam and compute the center of gravity of the stiffness, (See Fig. 5). From the eccentricity thus determined we compute the moment of eccentricity, computing now the moment of inertia of the stiffness units. We can compute the moment stress on the fibres at any point, or on the stiffness units at any point, located and identified as in Fig. 5. If we are regarding the stiffness units as fibres, the average K is to be regarded as the direct fibre stress, and the moment fibre stresses are to be added to or subtracted from the average K to give the true K at braced points along the exposure.

It seems reasonable to regard the axis of the beams as the location of the moment resistance, assembling the groups of joints acting along one line, where a number of beams are nearly in line, an approximation by inspection to the average line of action is probably quite close enough. Where a number of beams of small stiffness are over several feet apart, there is no useful purpose served in listing each as a separate force. The error in an approximate location of their center of gravity causes very small error in the important moments, and the moments in the shallow beams themselves are not important.

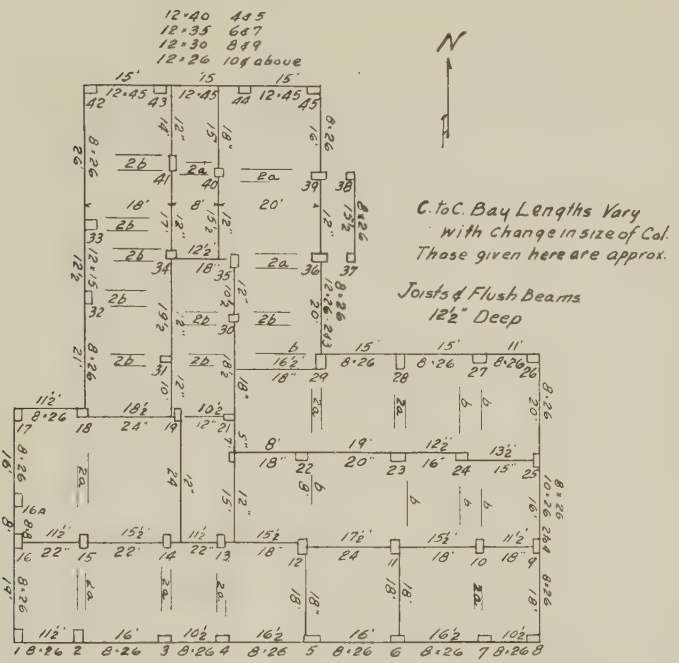


FIG. 4—EDDYSTONE BELMONT BLDG., SHERIDAN RD. AND BELMONT AVE. MCNALLY AND QUINN, ARCHITECTS. SMITH AND BROWN, ENGINEERS, INC. DIAGRAM OF FRAMING FOR DETERMINING STIFFNESS FACTORS FOR WIND STRESSES

The true K's for each group of joints having been determined we return to Tables 2 and 3 and multiply these values into the quantities $\frac{1}{R}$ to get the total moments at each of the joints.

These total moments are divided between the column sections above and below the floor, in proportion to the shears, except where there is great difference in the stiffness of the column above and below the floor. Where this disparity exists, the point of contra-flexure above is below the middle and the point of contraflexure below is below the middle; and the column below should be assigned moment in proportion to its stiffness. The column below should be assigned a moment somewhat greater than the total moment multiplied by the ratio of the shears. The ratio of the shears is found from the general stress diagram. For the second floor East and West, the shears above and below are 470,000 and 492,000. The proportion of moment above would be $\frac{470}{962} = .49$.

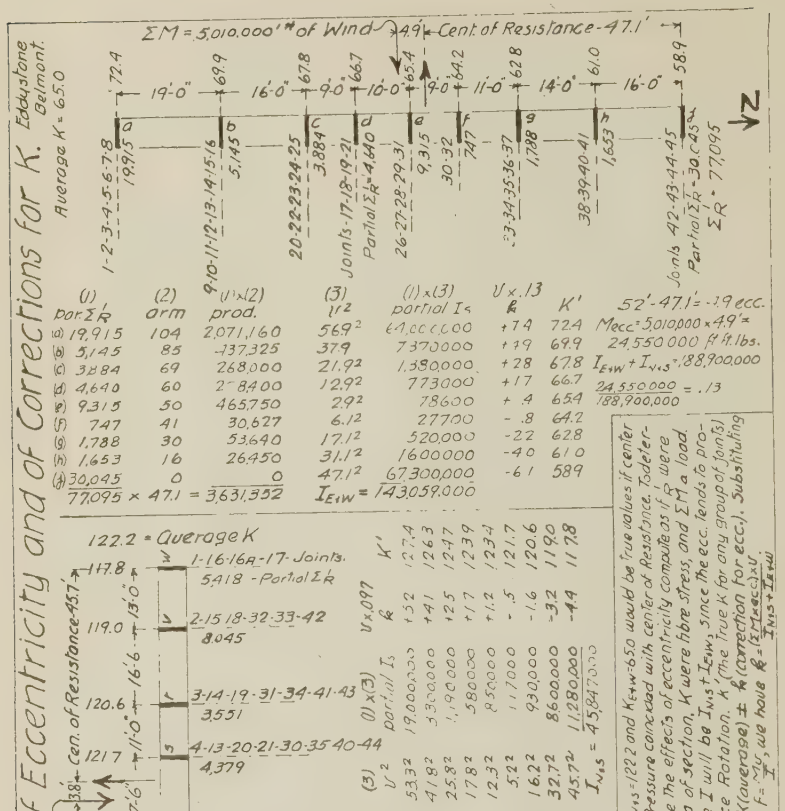


FIG. 5—CALCULATION OF ECCENTRICITY AND OF CORRECTIONS FOR K

TABLE 2—EDDYSTONE BELMONT BLDG., SECOND FLOOR—WIND EAST AND WEST

Col. Size	Col. Load	Mill.h $\frac{I}{L}$	Col. №	L	L'	$\frac{I_B}{1000L}$	$\frac{I_G}{1000L'}$	Million $\frac{I_B}{L}+\frac{I_G}{L'}$	Million xR	I R	true K	M in Joint	%	m Left	m' Rt	Rec. m Left	Rec. m' Rt
17x30		305	1	10.8'	–	2.2	–	455	763	1305	72.4	95	–	95	–	91	–
"		273	2	15.1'	10.8'	1.55	2.20	266	539	1860	"	135	58	56	79	61	83
"		77	3	9.2'	15.1'	2.54	1.55	245	322	3100	"	225	62	140	85	140	77
"		77	4	15.2'	9.2'	1.54	2.54	246	323	3090	"	224	62	84	140	87	140
17x34		51	5	14.6	15.2	1.61	1.54	317	368	2715	"	197	51	100	97	100	93
"		51	6	15.5	14.6	1.51	1.61	320	371	2695	"	195	52	95	100	89	100
17x30		77	7	9.2	15.5	2.53	1.51	247	324	3090	"	224	63	150	74	150	79
"		92	8	–	9.2	–	2.35	392	484	2060	"	150	–	–	150	–	150
18x32		240	9	–	10.7	–	.52	1920	2160	462	69.9	32	–	–	32	–	32
17x34		184	10	10.7	14.8	.52	.37	1120	1304	764	"	54	58	31	23	31	23
17x42		138	11	14.8	16.7	.37	.21	1710	1848	540	"	38	64	24	14	24	14
"		138	12	16.7	14.8	.21	.40	1640	1778	561	"	39	64	14	25	14	25
17x34		176	13	14.8	10.5	.40	.52	1095	1271	787	"	55	57	24	31	24	31
17x36		174	14	10.5	14.8	.52	.37	1120	1294	771	"	54	58	31	23	31	23
17x42		138	15	14.8	10.8	.37	.32	1120	1258	797	"	56	58	24	3	24	32
18x32		240	16	10.8	–	.52	–	1920	2160	463	"	32	–	32	–	32	–
17x30		300	17	11.5	–	2.06	–	486	786	1270	66.7	84	–	84	–	90	–
32x32		94	18	17.7	11.5	.37	2.06	411	505	1975	"	132	84	21	111	21	103
17x36		173	19	9.6	17.7	.59	.37	1030	1203	830	"	55	61	34	22	35	22
17x30		305	20	6.9	9.0	.79	.38	860	1165	858	67.8	58	68	39	19	41	19
"		90	21	–	9.6	–	.59	1680	1770	565	66.7	38	–	–	38	–	37
20x30		55	22	17.5	6.9	.37	.79	860	915	1093	67.8	74	68	24	50	24	47
"		63	23	14.7	17.5	.39	.37	1310	1373	730	"	49	51	25	24	25	24
17x34		54	24	12.7	14.7	.44	.39	1200	1254	798	"	54	53	29	25	29	25
17x36		196	25	–	12.7	–	.44	2270	2466	405	"	27	–	–	27	–	27
17x30		93	26	–	9.2	–	2.55	393	486	2055	65.4	135	–	–	135	–	132
"		80	27	9.2	14.1	2.55	1.66	237	317	3140	"	205	60	124	81	126	76
17x32		207	28	14.1	13.9	1.66	1.70	297	504	1980	"	130	51	64	66	70	72
24x30		88	29	13.9	23.0	1.70	.24	515	603	1660	"	109	87	96	13	89	13
17x34		59	30	23.0	15.0	.18	.28	2150	2209	451	64.2	29	61	11	18	11	18
17x30		82	31	15.0	23.0	.28	.24	1920	2082	480	65.4	31	54	17	14	17	14
17x36		173	32	11.0	–	.31	–	3200	3373	296	64.2	19	–	19	–	19	–
21x32		84	33	24.0	–	.18	–	5600	5684	176	62.8	11	–	11	–	11	–
26x26		84	34	12.1	23.0	.47	.24	1410	1494	670	"	42	66	28	14	27	14
17x30		302	35	15.8	12.1	.27	.47	1350	1652	606	"	38	63	14	24	15	25
"		88	36	3.3	15.8	.04	.27	3260	3348	299	"	19	88	2	17	2	17
17x17		517	37	–	3.3	–	.04	24,300	26,800	37	"	2	–	–	2	–	2
"		517	38	–	3.3	–	.04	26,300	26,800	37	61.0	2	–	–	2	–	2
17x30		88	39	3.3	18.6	.04	.23	3770	3,858	259	"	16	88	2	14	2	14
23x23		140	40	18.6	7.6	.23	.57	1,260	1,400	715	"	44	71	13	31	13	31
17x30		292	41	7.6	26.0	.37	.16	1,365	1,657	605	"	37	78	29	8	29	8
17x34		50	42	12.8	–	10.80	–	92	142	7015	58.9	420	–	420	–	373	–
18x32		67	43	13.7	12.8	10.0	10.8	48	115	8700	"	511	52	246	265	246	295
"		67	44	12.8	13.7	10.8	10.0	48	115	8700	"	511	52	265	246	280	246
17x30		85	45	–	12.8	–	10.8	92	177	5630	"	332	–	–	332	–	314
									ΣA =	77,095	ΣM =	5010					

Average $K = \frac{\Sigma M}{\Sigma A} = 65.0$

Mom. at Ends of Bms. corrected + or – by fraction – (Diff. in Moms)/2 x Sum

Correction at (1) 95–79 = 16 95+79 = 174 16/348 = .046 95 x .046 = 4 (minus)

TABLE 3—EDDYSTONE BELMONT BLDG., SECOND FLOOR—WIND NORTH AND SOUTH

Col. Size	Col. Load	Mill h / 2Ic	Col. No	L	L'	I_B /000L	I_B' /000L'	Million $I_B/L + I_B/L'$	Million $\times R$	ΣR	true K	M in Joint	%	m Left	m' Rt	Rev. m Left	Rev. m' Rt
17x30		90	1	—	17.6'	—	1.35	740	830	1210	1178	144	-	—	144	-	145
"		86	2	—	17.8'	—	.35	2850	2936	340	1190	40	-	-	40	-	40
"		280	3	—	18.5'	—	34	2950	3230	310	1206	37	-	-	37	-	38
"		280	4	—	18.4'	—	34	2950	3230	310	1217	37	-	-	37	-	38
17x34		230	5	—	16.8'	—	33	3020	3250	307	1234	38	-	-	38	-	38
"		230	6	—	16.8'	—	33	3020	3250	307	1247	38	-	-	38	-	38
17x30		280	7	—	17.2'	—	37	2700	2980	334	1263	42	-	-	42	-	43
"		320	8	—	17.1'	—	1.32	760	1080	925	1274	118	-	-	118	-	123
18x32		70	9	17.1'	14.5'	1.32	1.62	340	410	2440	1274	311	55	140	171	134	173
17x34		60	10	17.2'	15.6'	.37	.40	1300	1360	735	1263	93	52	45	48	44	47
17x42		32	11	16.8	24.0	.33	.36	1450	1482	674	1247	84	52	40	44	40	44
"		32	12	16.8	—	33	—	3020	3052	327	1234	40	-	40	—	40	—
17x34		58	13	18.4	20.0	34	23	1750	1808	553	1217	67	60	40	27	40	27
17x36		52	14	18.5	24.1	34	.27	1630	1682	593	1206	72	56	40	32	40	32
17x42		32	15	17.8	20.9	.35	.30	1530	1562	639	1190	76	54	41	35	41	35
18x32		70	16	17.6	6.5	1.35	.12	680	750	1333	1178	158	92	146	12	145	12
17x26		139	16A	6.5	14.1	.12	1.66	560	699	1425	1178	164	93	12	157	12	160
17x30		88	17	14.1	—	1.66	—	600	688	1450	1178	171	-	171	-	167	—
32x32		33	18	20.4	21.9	.30	1.07	730	763	1310	1190	156	78	35	121	35	118
17x36		52	19	24.1	9.4	27	.48	1330	1382	722	1206	88	64	32	56	32	54
17x30		88	20	20.0	6.3	.23	.48	1400	1488	673	1217	82	67	27	55	27	53
"		300	21	6.3	17.9	.48	32	1250	1550	645	1217	79	60	48	31	50	31
20x30		155	22	24.0	24.0	22	27	2030	2185	458	1234	56	55	25	31	25	31
"		152	23	24.0	25.0	.36	.25	1630	1782	560	1247	70	59	41	29	41	29
17x34		230	24	15.6	18.8	.40	.33	1370	1600	625	1263	79	55	43	36	44	36
17x35		55	25	14.5	19.2	1.62	1.22	352	407	2460	1274	315	57	180	135	178	129
17x30		320	26	19.2	—	1.22	—	820	1140	880	1274	112	-	112	—	117	—
"		280	27	18.8	—	.33	—	3020	3300	303	1263	38	-	38	—	38	—
17x32		73	28	25.0	—	.25	—	4000	4073	246	1247	31	-	31	—	31	—
24x30		56	29	24.0	18.9	.27	1.23	667	723	1380	1234	170	82	30	140	30	131
17x36		245	30	17.9	9.9	32	.47	1270	1515	660	1217	80	59	33	47	33	48
17x30		270	31	9.4	18.4	.48	.24	1380	1650	606	1206	73	67	49	24	51	25
17x36		49	32	21.9	11.0	1.07	2.12	314	363	2760	1190	329	66	111	218	113	207
21x32		122	33	11.0	25.3	2.12	.92	330	452	2220	1190	254	69	177	77	186	80
26x26		84	34	18.4	15.7	.24	.29	1880	1964	510	1206	63	55	29	34	28	34
17x30		88	35	9.9	18.0	.47	.31	1280	1368	720	1217	89	60	53	36	51	36
"		290	36	18.9	14.8	1.23	.18	706	996	1004	1234	125	87	109	16	116	16
17x17		466	37	—	14.8	—	1.60	625	1091	920	1239	114	-	-	114	-	114
"		466	38	14.8	—	1.60	—	625	1091	920	1239	114	-	114	-	114	-
17x30		290	39	14.8	15.3	.18	1.53	585	875	1140	1234	140	84	15	125	15	125
23x23		135	40	18.0	20.0	.31	.28	1700	1835	545	1217	66	53	35	31	35	31
17x30		86	41	15.7	16.0	.29	.28	1750	1836	545	1206	66	-	33	33	33	33
17x34		210	42	25.5	—	.92	—	1080	1290	776	1190	92	-	92	-	88	-
18x32		214	43	16.0	—	.28	—	3570	3784	265	1206	32	-	32	-	32	-
"		210	44	20.0	—	.28	—	3570	3780	265	1217	32	-	32	-	32	-
17x30		310	45	15.3	—	1.53	—	650	960	1040	1234	128	-	128	-	128	-
										$\Sigma 1/R =$	39378	$\Sigma M =$	4815				

Av. $K = \frac{\Sigma M}{\Sigma 1/R} = 122.2$

Revised m & m' obtained as for E & W stresses - See Table 2

The total joint moment is divided between the beams on either side of the column in the proportion of their values of $\frac{I}{1000L}$. In the column marked "%" is given the percentage found by dividing the larger $\frac{I}{1000L}$ by the sum. The correction of these moments is described later.

The moments here computed are those existing at the intersection of the center line of column with the center line of beam. These moments must be reduced to the face of the column and the face of the beam. Beam moments must be multiplied by the ratio one-half the span divided into one-half the span less one-half the column width. Similarly, the column moments must be multiplied by the ratio one-half the story height divided into one-half the story height less one-half the beam depth.

ACCURACY OF THIS METHOD

The results show that the assumption that points of contraflexure are at the middle of beams and columns is rarely exactly correct, and is often largely in error. After much recomputation of the joints adjacent to a beam whose end moments are unequal, we have found that if each moment is multiplied by a factor found by dividing the difference by twice the sum of the moments, the positive and negative correction found will make the corrected moments nearly correct.

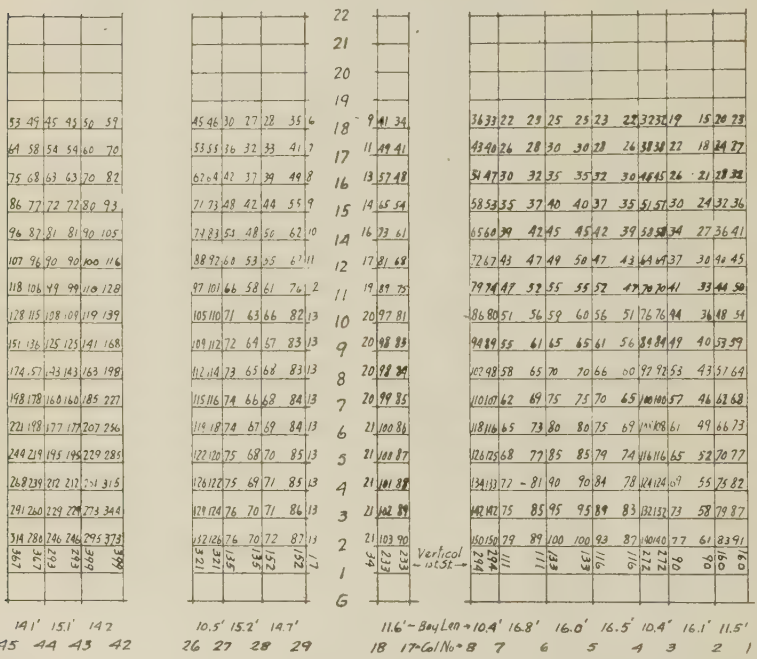
Where the column size changes, points of contraflexure above and below are not at mid-height, and in the rare case where the column wind stress at such a floor is critical, allowance must be made for additional moment at the upper end of the column below and at the upper end of the column above.

The wind moments for the various floors are computed for the full story height. In distribution of the moments the stiffness factors are based on beam lengths equal to the center to center of columns less one quarter the width of the two columns, and the column height is taken as floor to floor less one quarter the depth of two beams. In very short spans, whatever error this practice involves is magnified in effect. It is our practice, however, to make very short beams shallow.

FINAL SOLUTION FOR VERY UNSYMMETRICAL BUILDINGS

The stresses for Eddystone Belmont, shown on Tables 4 and 5, give a fairly accurate distribution of the moments and vertical

TABLE 4—EDDYSTONE BELMONT BLDG.—WIND EAST AND WEST

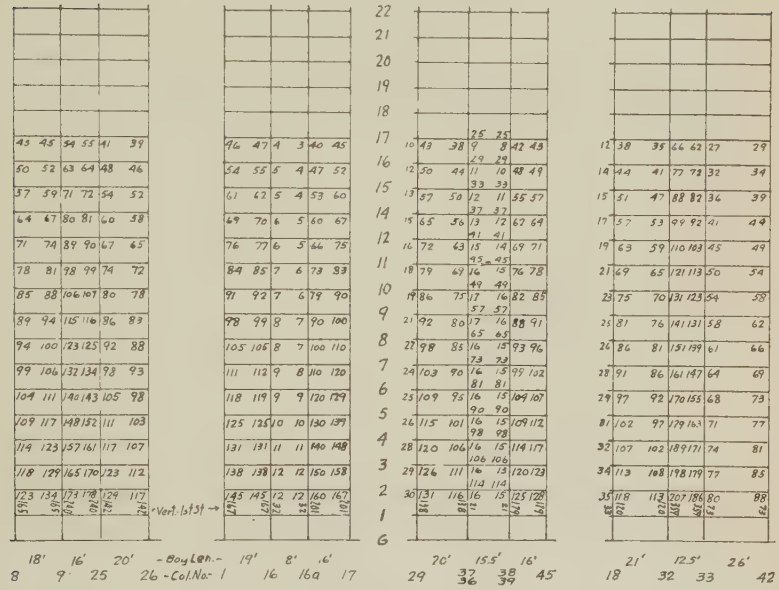


stresses in respect to bending of the beams and columns. Another element of deflection, that due to shortening of columns, must be taken into account.

It is clear that unless the exterior and first intermediate columns have a fibre stress from wind of the same sign and proportionate to their distance from the neutral axis of the bent, so that the floors remain in the same plane, secondary stresses are set up in the joints. These secondary stresses will remove vertical load from the exterior columns, and put loads of the same sign and of larger amount, in the intermediate columns. The major part of this transfer takes place in the upper third of the building, and in some cases the beams in the second bay of the bent require reinforcement for a few stories from the top.

In Eddystone Belmont, bay 17-18 is shown having moments of 103,000 ft. lb. and 90,000 ft. lb., and vertical loads in the columns of 199,000 lb. and 233,000 lb. It is quite certain that

TABLE 5—EDDYSTONE BELMONT BLDG.—WIND NORTH AND SOUTH



this bent cannot have the deflection which would result from the column deformation for these stresses. The stresses there shown are much too large. It is possible to make free hand adjustments of these stresses. To describe the adjustments which could properly be made in the stresses of this building is beyond the scope of this paper. I hope to compute a number of crooked buildings, and describe such stress adjustments, so that the designer will have available an example which will show the order of the stress aberrations, and can make an estimate instead of a guess, in the crooked problem before him.

ACCURACY OF THE CONVENTIONAL METHODS

This calculation has been prepared to show that the conventional methods not only leave a chance for error in determining what load comes to each bent, but involve in odd framing, errors in a single bent that are much too large. The labor of these calculations is considerable, but the engineer can readily deter-

mine what a conventional method would give him for moments in any one of these bents, and then determine for himself whether the error is tolerable.

Readers are referred to the JOURNAL *for June, 1931 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

Economics of Light Weight Concrete in Buildings

*Report of Committee 406, Use of Light Weight Concrete in Buildings**

BY FRANK A. RANDALL, AUTHOR-CHAIRMAN†

In the last few years several aggregates have been developed to produce lighter weight concretes, in addition to cinders which have been used extensively in some localities for many years.

The desirability of an aggregate to produce a lighter weight concrete is usually prompted by relative costs. With unit costs and strengths of the concretes equal, in general of course, the lighter concrete will produce the cheaper frame.

The present study is an attempt to show the comparative costs of the structural frames for concretes of various weights and costs. To cover all buildings in this comparison would be a large undertaking and it was necessary to introduce limitations which are stated in the following paragraphs. The author is of the opinion, however, that the data developed will apply without serious modification to the general problem.

The graphs and tables show how much more can be expended profitably to reduce the weight of concrete.

In using the data submitted herewith it should be borne in mind that an interior panel only is considered. For exterior panels, the effect of reduction in weight of the concrete floor construction is less than for interior panels, for two reasons. First, the spandrel girders and exterior columns carry the wall construction in addition to, usually, only one-half panel of floor construction and, hence, the dead weight of the floor is a smaller portion of the total load carried than for an interior panel.

*Prepared by Mr. Randall, this report was reviewed and in general approved before publication by the critic members of the committee to whom Mr. Randall makes acknowledgment: Frank Brown, of Smith & Brown, Chicago; Emil Praeger, New York City; W. S. Wolfe, of Smith, Hinchman & Grylls, Detroit.

†Structural Engineer, 205 West Wacker Drive, Chicago.

Second, particularly in high buildings, the sizes of the spandrel girders, and, sometimes, of the columns must be increased due to the wind load stresses; and this increase will be greatest for the lightest weight concrete, other conditions being the same.

A similar effect to that last mentioned would take place were the building subjected either to a live load or a wind load in excess of that for which it is designed, that is, the percentage of overstress would be greatest for the lightest weight concrete. The obvious answer is to design for the proper loadings, if possible. This is not a difficult matter as to live loads for the type of buildings considered, namely, residential or office. In the matter of wind loads, the judgment of the designer is required. Building ordinance requirements are minimum requirements, and it might be quite proper to use somewhat higher wind loads for the lighter weights of concrete than for the heavier weights of concrete in order to secure equivalent stiffness of the building.

One reason for the choice of panel size is that this size is used in Part Two, Cost Data in the *Handbook of Reinforced Concrete Building Design* of the American Concrete Institute.[1]

The lower limit of weights of concrete considered in this report is perhaps theoretical. Concrete weighing 60 lb. per cu. ft. has been used in 4-in. slabs spanning 5 ft. with a design live load of 100 lb. per sq. ft. and concrete weighing between 75 and 100 lb. per cu. ft. has been used in long span joist construction.

GENERAL ASSUMPTIONS

Type of Building. The buildings considered are of the residential or office type.

Loading. The loadings in pounds per sq. ft. of floor, used in the calculations, are as follows:

Live load	40
Partitions	20 (with no reduction of the live load on the beams and girders which is equivalent to an increase of the partition load on the beams and girders.)
Floor finish	5
Plaster, direct	5
Plaster on metal lath	10
Composition roofing	5

Panel Size. An interior panel 18 ft. square is considered.

[1] A. C. I., *Proceedings*, Vol. 24, p. 537. 1928.

Story Height. The story height is taken as 11 ft. floor to floor, for all stories including the basement.

Building Code. The unit stresses and regulations of the new proposed Chicago Building Code for fireproof construction are followed. This code embodies, substantially, the 1928 Joint Code and the American Institute of Steel Construction Code. Steel floor beams are designed as composite beams in accordance with the provisions of the new Chicago Code.

Concrete Stresses. The following unit compressive stresses in concrete are used as a maximum:

Floors	2000 lb. per sq. in.
Columns	3000 lb. per sq. in.

Unit Costs. The following unit costs of materials in place are assumed:

Reinforcement bars	$0.035	per lb.
Reinforcement mesh	0.045	per lb.
Spirals	0.05	per lb.
Structural steel	0.04	per lb.
Formwork	0.25	per sq. ft.
Concrete (substructure)	0.40	per cu. ft.
Excavation and backfill	2.70	per cu. yd.
Wood piles	25.00	each
Caissons (complete)	1.25	per cu. ft.
Plaster on joist construction, excess cost over plaster on solid slabs	0.01	per cu. ft. of building

VARIABLES

Weights of Concrete. Five weights of superstructure concrete are considered ranging from 50 lb. per cu. ft. to 150 lb. per cu. ft. as follows:

Type	Weight
A.	50 lb. per cu. ft.
B.	75 lb. per cu. ft.
C.	100 lb. per cu. ft.
D.	125 lb. per cu. ft.
E.	150 lb. per cu. ft.

Types of Floor Construction. Two types of floor construction are considered, viz.:

F.	Solid slabs 4 in. thick
G.	Steel form 20-in. joists

Types of Frame. Two types of frame are used, viz.:

H.	Concrete
J.	Steel

Building Heights. Six heights of building are considered ranging from five to thirty stories, and basement, viz.:

K.	5 stories and basement
L.	10 stories and basement
M.	15 stories and basement
N.	20 stories and basement
O.	25 stories and basement
P.	30 stories and basement

Foundations. Spread footings at 3000 lb. per sq. ft. are used under the 5 and 10-story buildings; wood piles at 25 tons each, under the 15 and 20-story buildings; and caissons on hard pan (50 ft. high) at 6 tons per sq. ft., under the 25- and 30-story buildings.

Cost of Superstructure Concrete. Three unit costs of superstructure concrete are considered, viz., thirty, forty and fifty cents per cu. ft. in place.

DETAIL ASSUMPTIONS

Stresses. The value of j was taken as equal to 0.875 for slabs, beams and girders. For slabs, the positive and negative moment coefficient was taken as 1/12; for beams, the positive coefficient was taken as 1/16 and the negative coefficient as 1/12. Unit shear was taken at 60 lbs. per sq. in. with the longitudinal reinforcement anchored. The allowable unit pressure under steel column bases was taken as 600 lbs. per sq. in.

Column Loads. The full panel area of 324 sq. ft. was used in computing the column loads, the load on the area occupied by the column being compensated by a reduction of one foot in the length of the column when computing the weight of the column, i. e., by using 10 ft. instead of 11 ft. for the length of the column.

Slabs. For the 4-in. solid slabs, the 18-ft. panel was divided into three equal spans. The slabs were designed for full continuity, using L as the distance center to center of the intermediate beams.

Beams and Girders. All beams and girders were made as shallow as possible, economically. Steel beams were designed for an L center to center of connections. The following allowances in center to center dimensions of steel framing were made: for beams, 6 in.; girders, 15 in. Concrete beams and girders were designed for the clear distance between supports.

Concrete Columns. Spirals were used in all cases. The sizes were kept constant in each group of five stories. For the type A and B concretes throughout all stories and for the type C, D and E concretes in the upper five stories, the column design was based on 2,000-lb. concrete; for the balance of the columns, 3,000-lb. concrete was used. Three spacers weighing ¾ lb. per ft. were used for core diameters from 10 in. to 19 in., inclusive, and four spacers for larger columns.

Steel Covering. The top flanges of steel beams were placed 3 in. below the top of the concrete slab, with 2-in. covering on the sides and bottoms. For the concrete slabs a minimum covering of ¾ in. was used on the bottom, and 1 in. on the top of the reinforcement steel. For the concrete joist construction a minimum covering of 1½ in. was used on the bottom and 1 in. on the top. For the upper five stories of the steel columns the minimum covering was taken at 2 in., and for the remainder a minimum covering of 3 in., in which latter case, only, the increased unit stresses for combination columns were used.

Weights of Structural Steel. To make allowance for details and rivets, steel beams were figured center to center in addition to the weight of the connection angles; and one foot was added to the column story height to include column splices, rivets and other details.

Excavation. For spread footings and concrete pile caps, 3 in. was included in the excavation outside of the finished concrete.

DESIGNS

The designs involved 20 roof panels, 20 floor panels, 620 columns and 120 foundations, all different. Consistency of design was kept in mind at all times in order to produce as truly comparable results as possible.

ESTIMATES

After the designs were completed, a careful estimate of all quantities was made to which were applied the unit costs. In the appendices are tabulated the quantities of material for each of the 120 conditions. Appendix 1 is a tabulation of the total quantities of materials in the floors, columns and foundations with the 4-in. solid slab and concrete frame. Appendix 2 is for

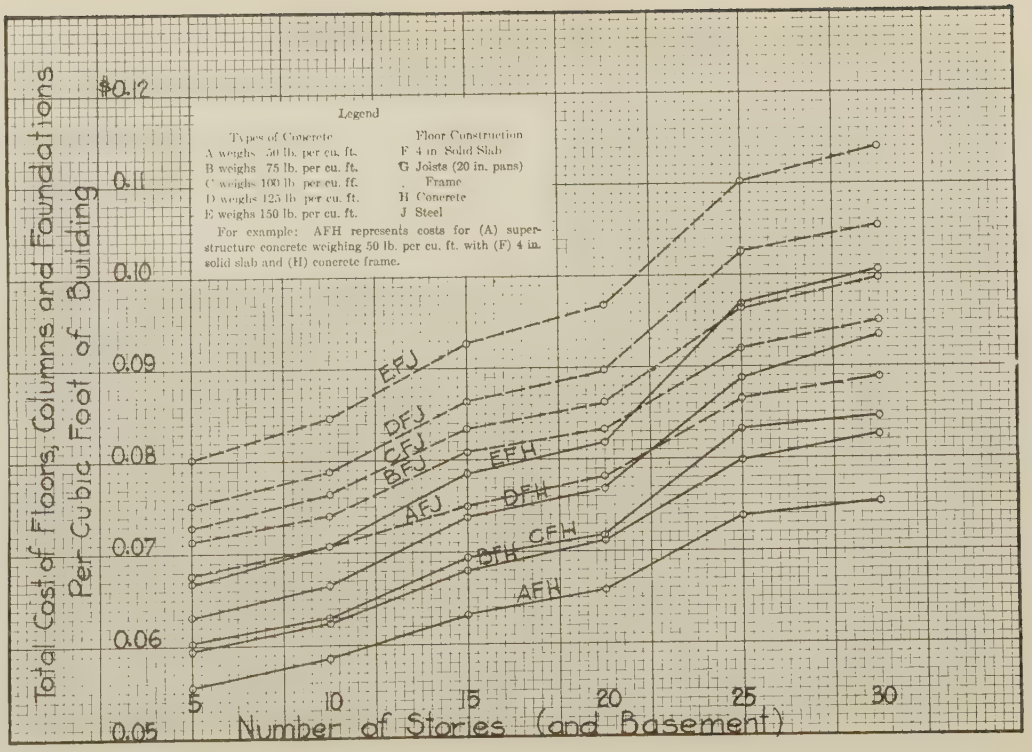


FIG. 1—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 30¢ PER CUBIC FOOT

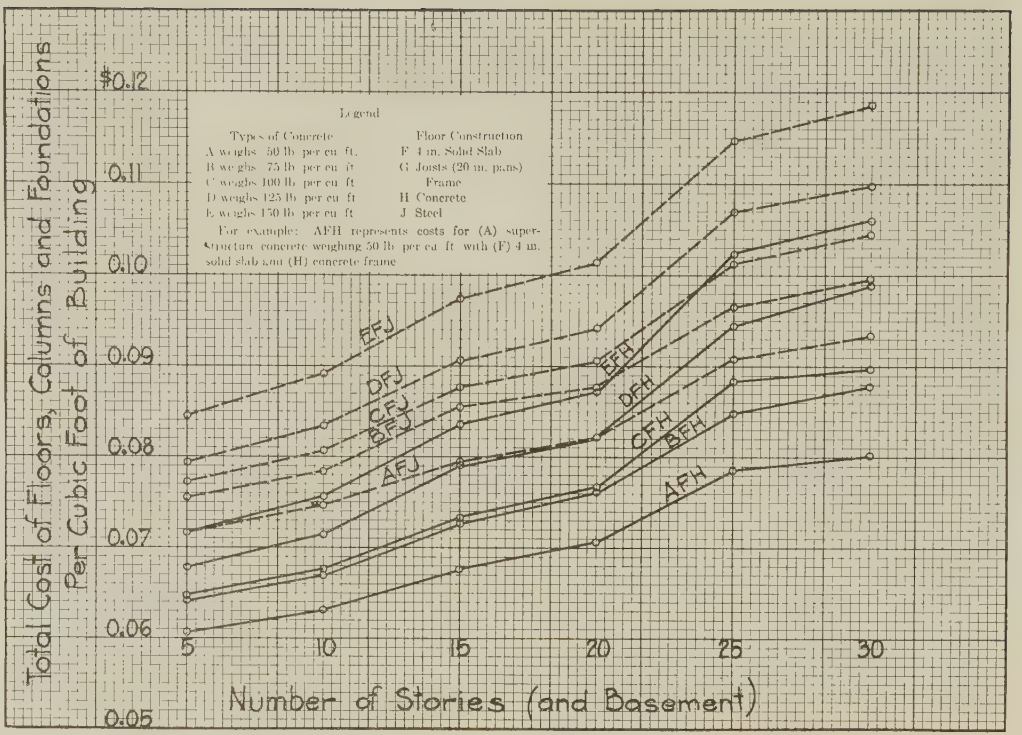


FIG. 2—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 40¢ PER CUBIC FOOT

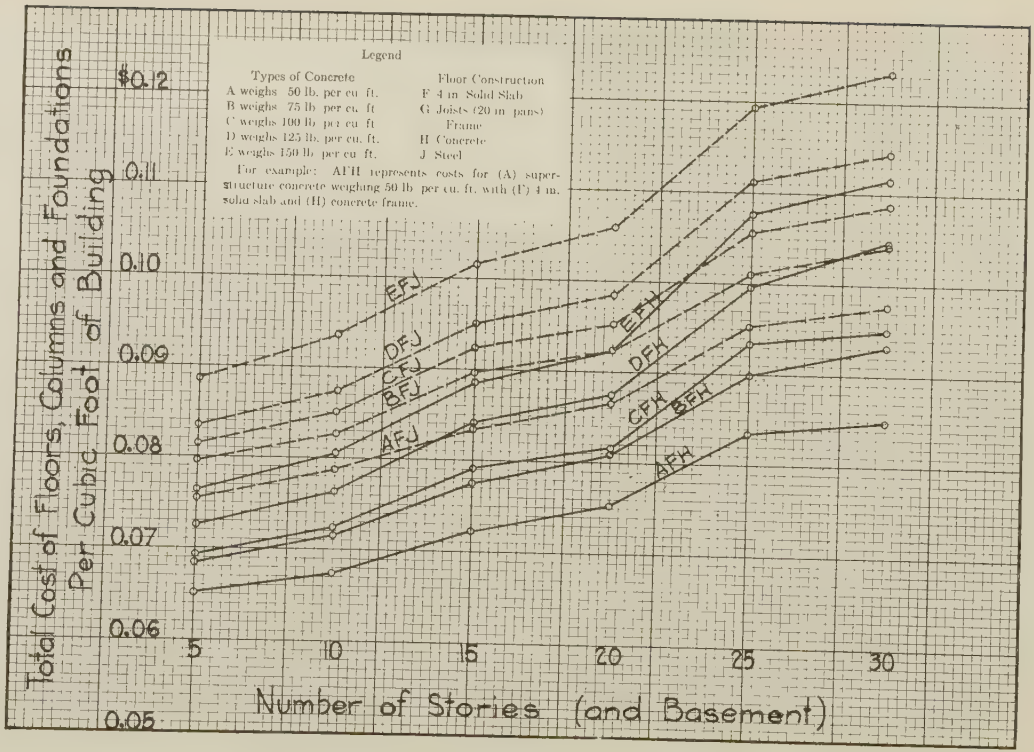


FIG. 3—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 50¢ PER CUBIC FOOT

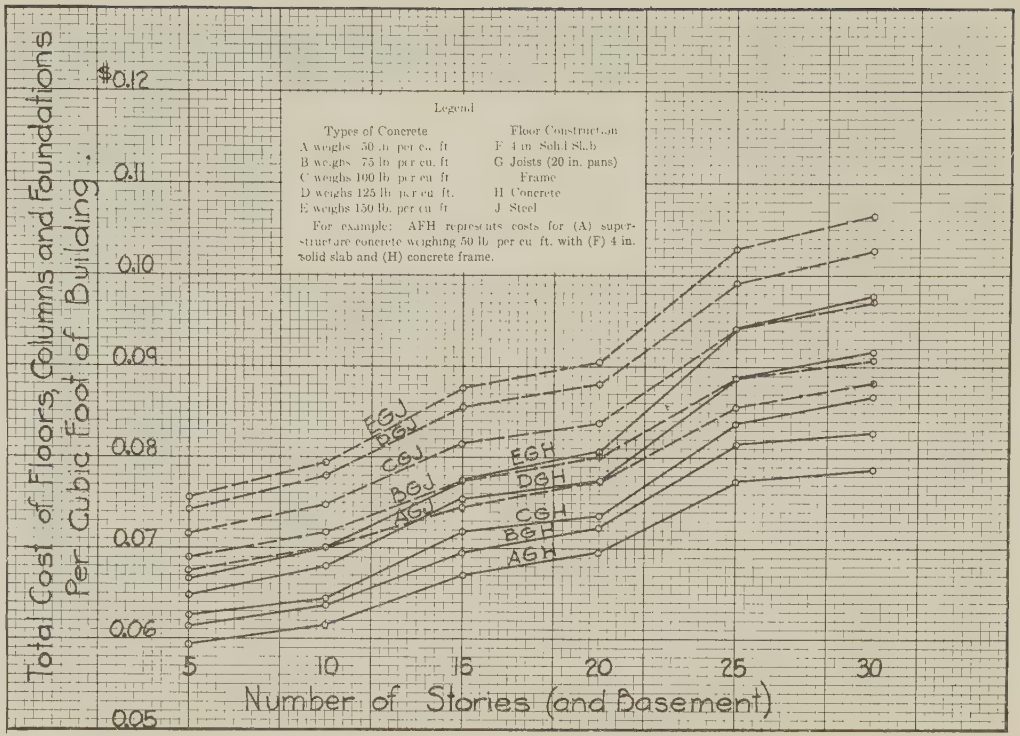


FIG. 4—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 30¢ PER CUBIC FOOT

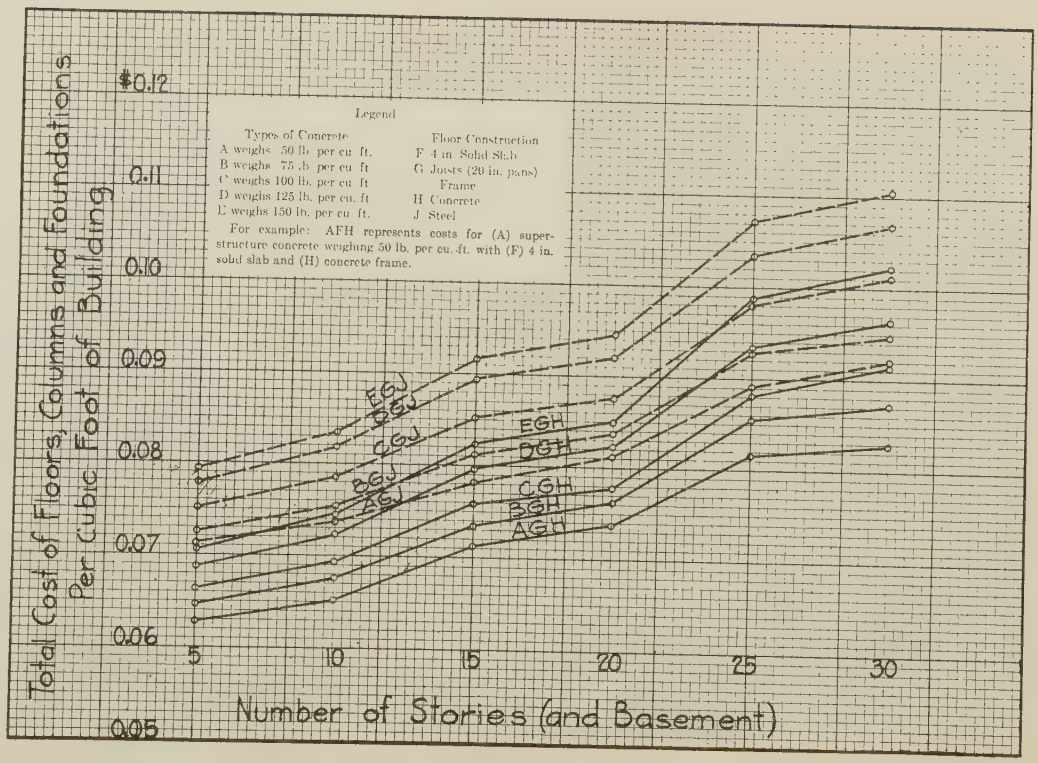


FIG. 5—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 40¢ PER CUBIC FOOT

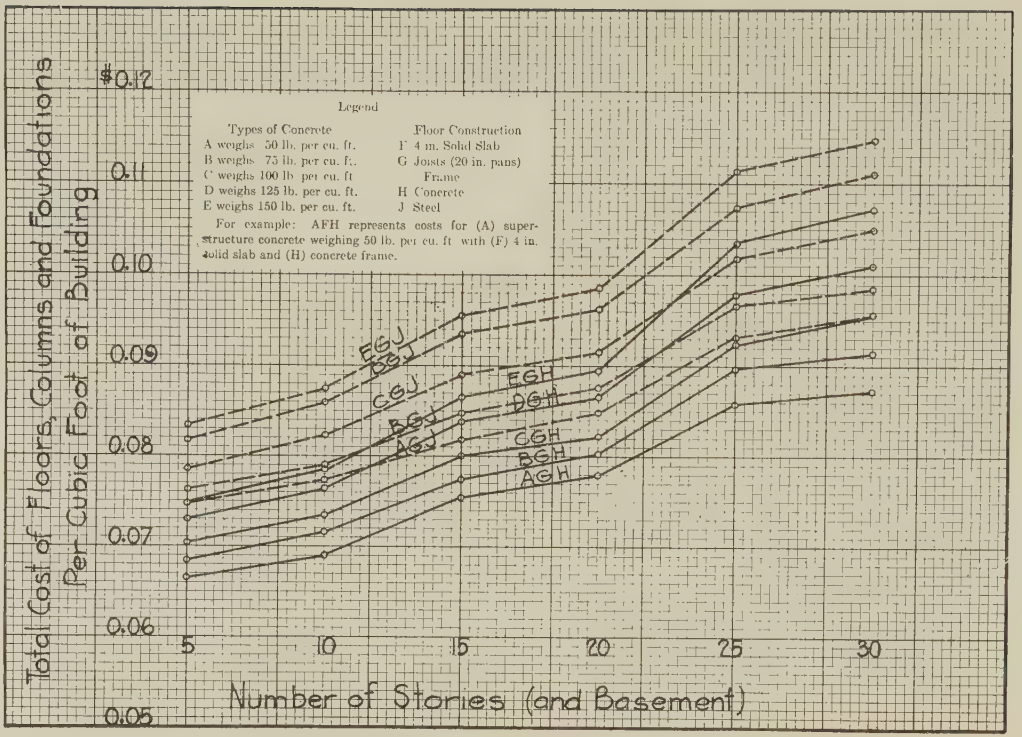


FIG. 6—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 50¢ PER CUBIC FOOT

TABLE 1—COST PER CUBIC FOOT OF BUILDING OF FLOORS, COLUMNS AND FOUNDATIONS

(SUPERSTRUCTURE CONCRETE AT 30¢ PER CUBIC FOOT)

Designation	Unit Weight of Superstructure Concrete Lbs. per cu. ft.	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
			Concrete Frame (H)				
			4-In. Solid Slab (F)				
AFH	50	$0.0562	$0.0586	$0.0631	$0.0659	$0.0738	$0.0752
BFH	75	.0595	.0623	.0680	.0712	.0798	.0827
CFH	100	.0602	.0629	.0694	.0717	.0832	.0845
DFH	125	.0631	.0664	.0739	.0769	.0889	.0934
EFH	150	.0669	.0708	.0785	.0819	.0969	.1004
			Joists (20-in. pans) (G)				
AGH	50	.0593	.0615	.0670	.0696	.0773	.0786
BGH	75	.0612	.0638	.0695	.0721	.0813	.0828
CGH	100	.0626	.0654	.0718	.0737	.0838	.0868
DGH	125	.0649	.0680	.0753	.0777	.0887	.0916
EGH	150	.0667	.0701	.0778	.0804	.0941	.0974
			Steel Frame (J)				
			4-In. Solid Slab (F)				
AFJ	50	$0.0676	$0.0706	$0.0750	$0.0781	$0.0865	$0.0889
BFJ	75	.0713	.0740	.0809	.0832	.0920	.0950
CFJ	100	.0729	.0763	.0833	.0861	.0965	.0997
DFJ	125	.0752	.0789	.0863	.0897	.1025	.1053
EFJ	150	.0801	.0848	.0928	.0969	.1102	.1140
			Joists (20-in. pans) (G)				
AGJ	50	.0675	.0700	.0744	.0773	.0854	.0881
BGJ	75	.0690	.0717	.0773	.0801	.0889	.0908
CGJ	100	.0715	.0749	.0814	.0838	.0941	.0971
DGJ	125	.0742	.0780	.0855	.0880	.0992	.1028
EGJ	150	.0755	.0794	.0876	.0903	.1029	.1063

joist construction with concrete frame; appendix 3, 4-in. solid slab with steel frame; and appendix 4, joist construction with steel frame. These tables are included in order that they may be used with unit costs different from those assumed in this paper.

Superstructure concrete of the various weights was kept separate from foundation concrete and the three different unit prices were used for the superstructure concrete, producing three total costs for each of the 120 buildings. These costs were then reduced to costs per cu. ft. of building, tabulated in Tables 1 to 3, inclusive, and which have been plotted in the following graphs:

Fig. 1 shows the cu. ft. (of building) cost for the 4-in. solid slabs for concrete (H) and steel (J) frame construction based on a unit cost of 30 cents per cu. ft. of the five weights of superstructure concrete.

Fig. 2 is the same as Fig. 1 excepting that the superstructure concrete is priced at 40 cents per cu. ft.

TABLE 2—COST PER CUBIC FOOT OF BUILDING OF FLOORS, COLUMNS AND FOUNDATIONS

(SUPERSTRUCTURE CONCRETE AT 40¢ PER CUBIC FOOT)

Designation	Unit Weight of Superstructure Concrete Lbs. per cu. ft.	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
			Concrete Frame (H)				
			4-In. Solid Slab (F)				
AFH	50	$0.0607	$0.0631	$0.0677	$0.0706	$0.0786	$0.0801
BFH	75	.0641	.0670	.0728	.0761	.0849	.0879
CFH	100	.0648	.0677	.0742	.0767	.0883	.0898
DFH	125	.0679	.0714	.0791	.0822	.0944	.0990
EFH	150	.0717	.0757	.0836	.0872	.1023	.1060
			Joists (20-in. pans) (G)				
AGH	50	.0629	.0652	.0712	.0738	.0816	.0829
BGH	75	.0648	.0676	.0734	.0761	.0855	.0871
CGH	100	.0664	.0694	.0759	.0779	.0881	.0913
DGH	125	.0689	.0722	.0796	.0822	.0933	.0963
EGH	150	.0707	.0744	.0822	.0850	.0988	.1022
			Steel Frame (J)				
			4-In. Solid Slab (F)				
AFJ	50	$0.0717	$0.0747	$0.0793	$0.0824	$0.0908	$0.0933
BFJ	75	.0755	.0784	.0854	.0877	.0965	.0966
CFJ	100	.0772	.0807	.0878	.0906	.1011	.1043
DFJ	125	.0794	.0833	.0907	.0941	.1070	.1099
EFJ	150	.0845	.0892	.0973	.1014	.1148	.1187
			Joists (20-in. pans) (G)				
AGJ	50	.0711	.0737	.0781	.0811	.0892	.0920
BGJ	75	.0726	.0754	.0811	.0838	.0928	.0947
CGJ	100	.0751	.0786	.0852	.0877	.0980	.1011
DGJ	125	.0780	.0819	.0895	.0920	.1033	.1069
EGJ	150	.0794	.0834	.0917	.0944	.1071	.1105

Fig. 3 is a similar set of curves with the concrete priced at 50 cents per cu. ft.

Fig. 4 is for the joist floor construction with the concrete and steel frames and the five weights of concrete costing 30 cents per cu. ft.

Fig. 5 is the same as Fig. 4 excepting the concrete is priced at 40 cents per cu. ft. and

Fig. 6 is the same for 50-cent concrete.

For those who prefer the tabular to the graphic, Tables 1, 2 and 3 show the same data as used in the preceding figures.

Table 1 shows the total cost per cu. ft. of building of the floors, columns and foundations, with superstructure concrete at 30 cents per cu. ft., for the five weights of concrete, the six story heights and for the two types of floor construction with both the concrete and steel frame.

TABLE 3—COST PER CUBIC FOOT OF BUILDING OF FLOORS, COLUMNS AND FOUNDATIONS

(SUPERSTRUCTURE CONCRETE AT 50¢ PER CUBIC FOOT)

Designation	Unit Weight of Superstructure Concrete Lbs. per cu. ft.	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
			Concrete Frame (H)				
			4-In. Solid Slab (F)				
AFH	50	$0.0651	$0.0676	$0.0723	$0.0754	$0.0835	$0.0850
BFH	75	.0687	.0717	.0776	.0810	.0900	.0931
CFH	100	.0694	.0724	.0791	.0816	.0934	.0950
DFH	125	.0727	.0764	.0842	.0875	.0998	.1046
EFH	150	.0765	.0807	.0888	.0925	.1078	.1116
			Joists (20-in. pans) (G)				
AGH	50	.0665	.0690	.0754	.0780	.0859	.0872
BGH	75	.0685	.0715	.0774	.0802	.0897	.0914
CGH	100	.0703	.0734	.0800	.0822	.0924	.0957
DGH	125	.0730	.0764	.0839	.0866	.0979	.1010
EGH	150	.0748	.0786	.0866	.0895	.1035	.1071
			Steel Frame (J)				
			4-In. Solid Slab (F)				
AFJ	50	$0.0758	$0.0789	$0.0835	$0.0867	$0.0952	$0.0977
BFJ	75	.0798	.0828	.0898	.0922	.1011	.1042
CFJ	100	.0815	.0851	.0923	.0952	.1057	.1089
DFJ	125	.0836	.0876	.0951	.0986	.1115	.1144
EFJ	150	.0888	.0937	.1018	.1060	.1194	.1233
			Joists (20-in. pans) (G)				
AGJ	50	.0747	.0773	.0819	.0849	.0931	.0958
BGJ	75	.0762	.0790	.0848	.0876	.0966	.0985
CGJ	100	.0787	.0823	.0890	.0915	.1019	.1050
DGJ	125	.0818	.0859	.0935	.0961	.1074	.1110
EGJ	150	.0833	.0875	.0957	.0986	.1113	.1148

Tables 2 and 3 contain the same data for superstructure concrete at 40 cents and 50 cents per cu. ft., respectively.

DEDUCTIONS

A study of these curves is of interest as disclosing:

1. The effect of increase in building height upon the cost of the structural frame, roughly 1/10 of a cent per cu. ft. of building per story for the range considered.

2. The greater effect of weight of the concrete in the higher buildings, roughly twice as much difference in the 30-story as in the 5-story building.

3. The comparative cost of concrete frame and steel frame, the difference in cost varying from about one cent to 1½ cents per cu. ft. of building.

4. The additional price that profitably may be spent for a lighter weight concrete in order to reduce the cost of the structural

frame. For example, it would be worth while to pay 10 cents more per cu. ft. to secure a reduction from 150-lb. concrete to 130-lb. concrete in a concrete frame building thirty stories high and a reduction to 115-lb. or 100-lb. concrete in a five story building, depending on whether the floor was of solid slab or joist construction. The savings are more favorable in the steel frame building.

We are hopeful that the data developed will prove of value.

(Tabulations constituting appendices 1 to 4 on following pages.)

Readers are referred to the JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

APPENDIX 1—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—4-IN. SOLID SLAB WITH CONCRETE FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	3377	7580	12358	18536	25306	33687
	Mesh	lbs.	836	1516	2196	2876	3556	4236
	Spirals	lbs.	421	941	1604	2496	3612	4951
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2767	5119.3	7526.7	10004.7	12289.0	14772.3
	Subst. Concrete	cu. ft.	79	172.9	148	283	..	..
	Excavation	cu. yd.	6.24	12.0	9.0	17.9	..	..
	Piles	one	..	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	955	1087
	Super. Concrete	cu. ft.	950	1778	2644	3549.7	4486.7	5453.4
75	Bars	lbs.	3887	8715	14232	21418	29465	39239
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	410	962	1731	2810	4095	5657
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2884	5350.3	7909.0	10497.3	12882.3	15502.3
	Subst. Concrete	cu. ft.	100.4	219.3	227	367	..	..
	Excavation	cu. yd.	7.7	14.7	14.3	23.1	..	..
	Piles	one	..	..	10	14	..	..
	Caissons	cu. ft.	..	..	..	..	1087	1342
	Super. Concrete	cu. ft.	977.1	1835.4	2745.0	3697.6	4687.8	5722.2
100	Bars	lbs.	4509	9796	15643	23193	31675	42201
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	402	950	1720	2798	4108	5715
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2804.3	5195.0	7674.7	10157.3	12465.7	14995.7
	Subst. Concrete	cu. ft.	119.9	251.4	283	367	..	..
	Excavation	cu. yd.	8.8	16.4	17.9	23.1	..	..
	Piles	one	..	..	12	16	..	..
	Caissons	cu. ft.	..	..	..	..	1342	1505
	Super. Concrete	cu. ft.	984.5	1858.2	2772.8	3734.3	4744.7	5783.4
125	Bars	lbs.	4799	10568	16856	25448	34983	47103
	Mesh	lbs.	986	1816	2646	3476	4306	5136
	Spirals	lbs.	461	1068	1991	3210	4746	6649
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2862.3	5312.0	7864.0	10430.0	12755.7	15352.3
	Subst. Concrete	cu. ft.	149.1	323.1	367	490.0	..	..
	Excavation	cu. yd.	10.6	20.2	23.1	30.4	..	..
	Piles	one	..	..	14	19	..	..
	Caissons	cu. ft.	..	..	..	..	1505	1936
	Super. Concrete	cu. ft.	1032.5	1957.5	2938.4	3969.6	5053.0	6170.3
150	Bars	lbs.	5816	12818	20332	30387	41694	55829
	Mesh	lbs.	991	1826	2661	3496	4331	5166
	Spirals	lbs.	469	1121	2160	3506	5206	7283
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2988.3	5548.7	8196.3	10881.0	13304.7	16024.7
	Subst. Concrete	cu. ft.	172.9	378.9	367	537	..	..
	Excavation	cu. yd.	12.0	23.2	23.1	32.6	..	..
	Piles	one	..	..	16	21	..	..
	Caissons	cu. ft.	..	..	..	..	1779	2143
	Super. Concrete	cu. ft.	1032.6	1951.7	2928.7	3957.9	5045.1	6185.8

APPENDIX 2—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—JOISTS WITH CONCRETE FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	4114	8902	14265	21028	28383	37349
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	397	917	1580	2472	3588	4927
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2342.0	4334.3	6381.7	8499.7	10424.0	12547.0
	Subst. Concrete	cu. ft.	79.0	172.6	148.0	283.0	..	..
	Excavation	cu. yd.	6.24	12.0	9.0	17.9	..	..
	Piles	one	..	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	955	1087
	Super. Concrete	cu. ft.	781.1	1467.6	2387.6	3151.3	3946.3	4770.9
75	Bars	lbs.	4775	10358	16603	24364	33192	43738
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	382	924	1660	2709	3997	5544
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2367.0	4383.3	6494.0	8617.3	10564.0	12720.7
	Subst. Concrete	cu. ft.	94.6	209.8	227.0	346.0	..	..
	Excavation	cu. yd.	7.3	14.1	14.3	22.1	..	..
	Piles	one	..	..	10	13	..	..
	Caissons	cu. ft.	..	..	..	..	1087	1232
	Super. Concrete	cu. ft.	793.6	1495.1	2248.0	3044.2	3873.4	4734.6
100	Bars	lbs.	5093	10831	17148	25068	34116	44889
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	375	885	1621	2669	3955	5516
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2377.3	4411.0	6529.7	8649.0	10584.0	12754.0
	Subst. Concrete	cu. ft.	113.4	251.4	283.0	367.0	..	..
	Excavation	cu. yd.	8.4	16.4	17.9	23.1	..	..
	Piles	one	..	..	12	15	..	..
	Caissons	cu. ft.	..	..	..	..	1211	1505
	Super. Concrete	cu. ft.	820.1	1556.0	2333.0	3153.1	4010.3	4911.7
125	Bars	lbs.	5439	11721	18506	27230	36940	48887
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	398	973	1792	2952	4377	6130
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2432.0	4508.0	6682.3	8879.7	10845.7	13070.7
	Subst. Concrete	cu. ft.	126.6	284.9	346.0	445.0	..	..
	Excavation	cu. yd.	9.6	18.3	22.1	28.2	..	..
	Piles	one	..	..	13	17	..	..
	Caissons	cu. ft.	..	..	..	..	1365	1652
	Super. Concrete	cu. ft.	864.5	1635.8	2461.8	3335.5	4260.4	5217.9
150	Bars	lbs.	5763	12495	19746	29354	40008	53332
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	412	1040	1994	3243	4820	6776
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2466.0	4562.7	6757.7	8968.7	10942.7	13197.7
	Subst. Concrete	cu. ft.	149.1	335.7	367.0	490.0	..	..
	Excavation	cu yd.	10.6	21.0	23.1	30.4	..	..
	Piles	one	..	..	15	19	..	..
	Caissons	cu ft.	..	..	..	..	1626	1961
	Super. Concrete	cu. ft.	881.0	1665.8	2507.2	3398.0	4346.2	5340.8

APPENDIX 3—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—4-IN. SOLID SLAB WITH STEEL FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	184	443	221	335	..	..
	Mesh	lbs.	836	1516	2196	2876	3556	4236
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	9701	19498	30745	43694	58538	74814
	Formwork	sq. ft.	2786.5	5173.2	7594.5	10070.8	12344.3	14791.8
	Subst. Concrete	cu. ft.	79.0	181.3	148	283	..	..
	Excavation	cu. yd.	8.9	17.2	12.0	22.4	..	..
	Piles	one	..	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	971.6	1190.9
	Super. Concrete	cu. ft.	872.9	1635.0	2417.3	3218.0	4035.7	4853.8
75	Bars	lbs.	218	534	335	394	..	..
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	10074	20381	32762	46957	63338	81517
	Formwork	sq. ft.	2918.0	5350.5	7899.8	10425.8	12752.5	15283.5
	Subst. Concrete	cu. ft.	100.4	219.3	283.0	367.0	..	..
	Excavation	cu. yd.	10.9	20.0	22.4	28.2	..	..
	Piles	one	..	..	11	14	..	..
	Caissons	cu. ft.	..	..	..	..	1087.3	1341.9
	Super. Concrete	cu. ft.	910.2	1712.0	2531.1	3369.9	4223.6	5079.8
100	Bars	lbs.	277	655	335	394	..	..
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	10720	21900	35363	50955	68907	88979
	Formwork	sq. ft.	2890.5	5364.5	7910.0	10436.8	12771.2	15306.1
	Subst. Concrete	cu. ft.	119.9	251.4	283.0	367.0	..	..
	Excavation	cu. yd.	12.4	22.2	22.4	28.2	..	..
	Piles	one	..	..	12	16	..	..
	Caissons	cu. ft.	..	..	..	..	1232.0	1505.3
	Super. Concrete	cu. ft.	916.5	1722.4	2546.8	3389.1	4249.9	5112.5
125	Bars	lbs.	306	782	384	511	..	..
	Mesh	lbs.	986	1816	2646	3476	4306	5136
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	11487	23461	37776	54796	74202	95987
	Formwork	sq. ft.	2908.8	5395.2	7943.3	10496.5	12801.0	15340.2
	Subst. Concrete	cu. ft.	140.5	296.5	346.0	445.0	..	..
	Excavation	cu. yd.	14.0	25.5	27.0	34.5	..	..
	Piles	one	..	..	13	18	..	..
	Caissons	cu. ft.	..	..	..	..	1505.3	1779.5
	Super. Concrete	cu. ft.	905.1	1699.8	2508.2	3335.6	4182.1	5029.5
150	Bars	lbs.	392	908	394	553	..	..
	Mesh	lbs.	991	1826	2661	3496	4331	5166
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	13496	27790	44736	64612	87240	112614
	Formwork	sq. ft.	2935.5	5438.8	8003.5	10579.2	12872.0	15422.8
	Subst. Concrete	cu. ft.	156.7	349.6	367.0	537.0	..	..
	Excavation	cu. yd.	15.3	29.1	28.2	40.0	..	..
	Piles	one	..	..	15	20	..	..
	Caissons	cu. ft.	..	..	..	..	1625.8	1971.1
	Super. Concrete	cu. ft.	926.7	1737.1	2562.5	3405.7	4270.0	5132.8

APPENDIX 4—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—JOISTS WITH STEEL FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	1966	3755	5083	6737	7942	9482
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	6588	13401	21820	31668	43449	56799
	Formwork	sq. ft.	2366.5	4393.2	6456.2	8574.2	10487.7	12575.2
	Subst. Concrete	cu. ft.	79.0	181.3	148.0	283.0	..	..
	Excavation	cu. yd.	8.9	17.2	12	22.4	..	..
	Piles	one	.	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	954.6	1190.8
	Super. Concrete	cu. ft.	767.0	1439.1	2131.4	2843.4	3570.5	4298.5
75	Bars	lbs.	2140	4124	5601	7346	8642	10322
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	6535	13378	22110	32811	45289	59621
	Formwork	sq. ft.	2344.5	4350.8	6418.2	8479.8	10355.7	12417.5
	Subst. Concrete	cu. ft.	94.6	209.8	227.0	346.0	..	..
	Excavation	cu. yd.	10.3	19.2	18.4	27	..	..
	Piles	one	..	..	10	13	..	..
	Caissons	cu. ft.	..	..	..	..	1068.7	1232.0
	Super. Concrete	cu. ft.	764.1	1435.9	2125.2	2832.2	3555.7	4280.0
100	Bars	lbs.	2404	4643	6232	8176	9667	11552
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	7624	16027	26213	38858	53463	70271
	Formwork	sq. ft.	2384.5	4425.5	6536.0	8627.8	10522.5	12622.2
	Subst. Concrete	cu. ft.	113.4	251.4	283	367	..	..
	Excavation	cu. yd.	11.7	22.2	22.4	28.2	..	..
	Piles	one	..	..	12	15	..	..
	Caissons	cu. ft.	..	..	..	..	1210.9	1480.3
	Super. Concrete	cu. ft.	777.1	1459.2	2160.4	2879.6	3613.9	4353.0
125	Bars	lbs.	2790	5429	7298	9640	11344	13559
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	8228	17248	28478	42102	58407	76855
	Formwork	sq. ft.	2387.8	4438.8	6552.3	8568.8	10534.2	12635.8
	Subst. Concrete	cu. ft.	126.6	284.9	346	445	..	..
	Excavation	cu. yd.	13	24.6	27	34.5	..	..
	Piles	one	..	..	13	17	..	..
	Caissons	cu. ft.	..	..	..	..	1341.9	1652.3
	Super. Concrete	cu. ft.	814.1	1536.9	2274.9	3031.6	3805.4	4582.1
150	Bars	lbs.	2837	5557	7347	9679	11383	13598
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	8408	17871	29797	44292	61621	81399
	Formwork	sq. ft.	2411.8	4473.2	6611.3	8724.5	10601.3	12715.5
	Subst. Concrete	cu. ft.	140.5	296.5	367	445	..	..
	Excavation	cu. yd.	14	25.5	28.2	34.5	..	..
	Piles	one	..	..	14	18	..	..
	Caissons	cu. ft.	..	..	..	..	1480.3	1779.5
	Super. Concrete	cu. ft.	832.7	1572.1	2325.2	3097.3	3886.3	4677.6

Discussion of the Paper by Helmer Swenholt:

"Design and Construction of Bonnet Carre Spillway"*

BY C. E. GRUNSKY†

There are some features of the Bonnet Carre Spillway described by Captain Swenholt, which might well be further elucidated for the benefit of the engineering profession. Among these are the following:

Why, for example, in view of the infrequent occurrence of the large discharge, which is anticipated about once in five years, was the spillway not made longer? A maximum overflow depth of 4 or 5 ft. would have been more desirable than 8 ft. Moreover it might even have been possible to secure the desired maximum flow at elevation 24 ft. over a weir crest without controlling the flow with gates or needles and yet without excessive premature functioning of the spillway. Operation costs could have been thereby materially reduced. The results of any studies made of such an arrangement, if topographic features do not preclude it, would be a welcome addition to the paper.

Undoubtedly there must be good reason for supporting the concrete weir structure on piles. Under ordinary conditions this would seem to the writer an undersirable arrangement because the separation of the ground from the base of the pile supported structure by settlement may take place and thereby the piping out of the soil from under the structure is facilitated and maximum uplift is invited. Generally speaking and entirely without personal knowledge of, and, perhaps, without application to conditions at the spillway site, it may be stated that such structures should preferably rest directly with their full weight upon the underlying ground. It is good practice moreover to add weight

†Consulting Engineer, San Francisco.
*Journal A. C. I., November 1930; A. C. I. *Proceedings*, Vol. 27, p. 243.

by providing an overhead sand box if there is considerable difference in elevation between the high water on one side of the spillway and the low water stage on its other side, so that by heavy loading the underlying material will be compacted and the danger of piping out the support will be reduced. Furthermore, it may frequently be advisable to place such a structure on a layer of pervious material, such as sand, so that percolating waters which get past the upper lines of sheeting and of cutoff walls may find easy passage to a down-stream outlet.

The line of steel sheet piling close to the upstream edge of the weir affords it a maximum amount of support along or near its upstream edge. Longitudinal cracking of the floor along the trench feature, shown at the face of the weir, and into the piers may result if there should be more settlement of the piles than of the sheeting. Some explanation would be desirable as to this location of the sheeting in preference to a location under the crest line of the weir where it would have been equally effective in checking the underflow. Again referring to ordinary conditions and to a structure without pile support, the sheeting would then, it seems to the writer, more logically be placed just above the weir structure with concrete resting against it and not upon it.

The reason for the second line of steel piling at the stilling basin is not clear. Something at that point in the nature of protection against under-cutting in case of a failure of the flexible concrete mat and rock fill may be desirable, but the sheeting, as described, functioning to retard underflow, will build up pressure under the fore-apron and this pressure, unless measures for adequate relief have been provided, might float the fore-apron. Under an extreme, and probably unwarranted assumption, that water might find more obstruction to flow at the down-stream line of sheeting than at the upstream line, the uplift due to a difference in head of some 12 feet just before a release of the needles could easily be sufficient to lift and crack the apron.

The engineering profession is under particular obligations to Captain Swenholt for his clear and concise description of this spillway which, because of the great volume of water to be handled, marks a forward step in flood control practice. Later accounts of its effect on flood stages will be eagerly looked for by all hydraulic engineers.

Discussion of Paper by L. J. Mensch:

"Composite Columns"*

BY DAVID MOLITOR†

Some years ago the writer reviewed an article published in 1911, by Dr. F. Emperger on "A new use of cast iron in columns and arch bridges" describing a rather exhaustive series of tests from which he deduces the following conclusions:

1. That the strength of reinforced concrete columns was increased only slightly by vertical bars, while the hooping was mainly responsible for the enhanced carrying capacity.
2. Composite columns, consisting of a steel or cast iron core and encased in a concrete envelope with vertical reinforcement and hooping, could be designed to give exceptional results but depend mostly on the hooping for their greater strength.
3. Filling the interior of the cast iron cylinder with concrete materially diminished the ultimate strength of the encased column. A marked reduction in strength also, followed when octagonal or square spirals were substituted for the round form.

The form of column giving the highest strength according to Emperger, consisted of a hollow cast iron cylinder surrounded by a concrete encasement containing very little vertical reinforcement, but considerable spiral steel, as shown in Fig. 1.

Without attempting any further description of these experiments, it is at once apparent that the author's test columns differ from those of Emperger in that the hollow core feature was disregarded and only solid sections were tested, giving some rather erratic results depending on whether the hooping was capable or incapable of acting.

From all the test data on composite columns examined by the writer, it would appear that the element of weakness in such columns, as in ordinary hooped concrete columns, is due to insufficient hooping material to resist lateral bursting. No

*Journal A. C. I., Nov. 1930, A. C. I. *Proceedings*, Vol. 27, p. 263.
†Structural Engineer, Detroit.

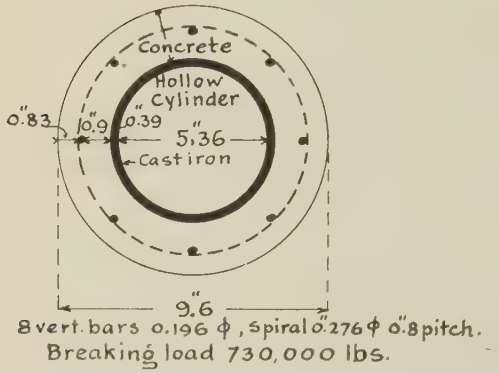


FIG. 1

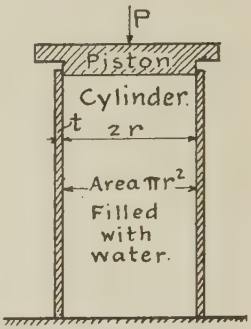


FIG. 2

matter how much core material is supplied, if the hoop strength is not properly provided, the column fails under a low test load.

To illustrate this point, let the cylinder Fig. 2, represent a hollow column as of spirally welded pipe, filled with a liquid, as water, and supported on a base with water-tight connection. On top of the cylinder is a piston acting as a column cap and fitting into the cylinder with water-tight joints but not otherwise transferring any vertical load to the tube. Disregarding details and assuming absolutely tight joints, this is at once a column capable of sustaining any load P by stressing the cylinder only in hoop tension. Let F represent the allowable working tensile stress in pounds per sq. inch, and referring to Fig. 2.

Then the unit pressure on the cylinder wall would be $p = \frac{P}{A} = \frac{P}{\pi r^2}$ and the hoop tension pr would require a thickness of metal $t = \frac{pr}{f} = \frac{P}{f\pi r}$.

The required steel area $= \frac{2\pi r\, P}{f\pi r} = \frac{2P}{f}$ = constant for any given load P.

Hence for $P = 1{,}000{,}000$ lbs. and $f = 18{,}000$ lbs., the steel area would be 112 sq. ins., requiring a cylinder of any diameter with a corresponding thickness t to give this area. Thus for $t = 1$ inch, the diameter would be 35 inches, and the pressure on the water would be 1000 pounds per sq. inch.

In this example we are dependent on a frictionless liquid which transmits the full pressure to the cylinder wall. Now substitute concrete for the liquid and note that most of the lateral pressure is absorbed by internal friction or stress, and only a small fraction of the unit pressure will remain to exert hoop tension, although in the absence of the tension element the concrete core would burst when the load P attains a certain magnitude.

Therefore, there is no reason why such a column could not be designed for any unit load up to or even exceeding the crushing strength of the concrete and require only a relatively thin enclosing steel cylinder, or spiral, subjected only to tensile stress.

According to this reasoning, which accords with the results obtained from tests, it would appear that the capacity of a composite column does not depend on the amount of vertical steel contained therein, but on how well the concrete is confined within the hooping or tensile shell surrounding the concrete core.

The Emperger cast iron columns substantiate this conclusion to a marked degree, for when the hollow c.i. cylinder is filled with concrete, the strength is materially reduced, because the cast iron shell is not suitable to resist hoop tension.

The best solution of this problem (still to be verified by tests) might be found by providing sufficient steel in the form of heavy spiral steel without appreciable vertical steel, designed entirely to resist tension. For this purpose flat bars would be preferable to round hooping bars and the maximum pitch of the spiral should be such as to prevent the flow of concrete through the open space and yet not so small as to prevent good bond between the core and the insulation.

The several test columns described by Mr. Mensch do not reveal any definite consistency in behavior and we must conclude that the underlying law for the strength of composite columns is still somewhat doubtful.

For columns consisting essentially of a steel core with concrete encasement, the increased strength which may exist is undoubted-

ly due to increased stiffness of the steel rather than to direct carrying capacity of the concrete.

For columns consisting of a concrete core surrounded by steel hooping, the increased strength is a direct function of the amount of hooping provided, because it is the hooping which directly opposes the lateral bursting pressure and prevents failure by diagonal tension.

Readers are referred to the JOURNAL *for June, 1931, for further discussion, including author's closure.*

The American Concrete Institute and Concrete Research*

BY DUFF A. ABRAMS

The American concrete institute is a working organization which depends for results solely on the voluntary effort of the members who give so freely of their time and experience in the work of its committees and in the preparation of papers and reports for the benefit of all.

Probably the most outstanding accomplishment of the year 1930 was the organization of many new technical committees and the increasing activity of the committees already organized. The Institute now has 71 technical committees made up of 218 individuals. Each committee has been assigned a well-defined field for study and report.

The committees are grouped under the following headings:

Research
Materials
Engineering Design
Architectural Design
Specifications
Construction
Shop Manufacture
Use Requirements
Joint Efforts

It is the purpose to cover the entire field of concrete materials and usage and to bring to the members the most reliable data, experience or opinion bearing on all phases of concrete work.

The method of appointing an author-chairman who is responsible for the initial form of the report, has been most successful. Before a report is submitted to the Institute it is reviewed by two or more critic members who are selected on account of special qualifications for the work at hand.

*Address of the President, 27th Annual Convention of the American Concrete Institute, Milwaukee, February 26, 1931.

Another outstanding event of 1930 was the completion of the first full year of publication of the new JOURNAL. Publication of the JOURNAL was begun in November 1929; the first 8 issues comprise Vol. 26 of the *Proceedings*. These 8 issues contained 1238 pages, by far the largest number of pages published in any year. Seven issues of the current JOURNAL, September 1930 to March 1931, contain 1294 pages.

The JOURNAL has been highly successful in meeting the needs of the Institute members. The Institute is no longer dependent upon one annual forum for discussion of its problems; it now has ten forums in addition to the annual convention. The papers published in the JOURNAL are of a high order of merit and will, no doubt, place this publication in the first rank of journals devoted to concrete.

An examination of the published *Proceedings* of the Institute from 1905 to 1931 shows some interesting and significant changes in the center of interest of the members.

A classification of the material in the proceedings for two-year periods at about five year intervals gives the following results:

	Per Cent of Total Pages of Papers and Reports					
Years Averaged	Re-search	Design	Con-struction	Speci-fication	Products Plant	Misc.
1905 and 1906	4	3	31	13	39	9
1910 and 1911	14	11	52	17	6	0
1916 and 1917	34	9	36	13	4	4
1920 and 1921	34	19	44	5	4	1
1925 and 1926	40	12	29	12	5	2
1930* and 1931*	50	18	17	13	2	0

*JOURNAL, Nov. 1929 to Feb. 1931

At the beginning most attention was given to products plants, construction and specifications, with little space devoted to design and research. This condition has undergone a gradual change during the quarter-century of the Institute's active life, until the present publications exhibit a predominant interest in research with the remainder nearly equally divided between design, construction and specifications. Since the beginning of publication of the JOURNAL 50 per cent of papers and reports have been devoted to research. In arriving at this figure *Abstracts* and *News Letters* were not considered.

Probably the most interesting phase of this change is that it has apparently come about without aim or guidance on the part of those responsible for the direction of the Institute, but rather as an unconscious adaptation of the work of the Institute to the greatest needs of its members.

It is my belief that the best interests of the Institute demand a still greater recognition of the function of research in our field to the end that the Institute may become the recognized clearing house for concrete research.

What is the field for research in the concrete industries? It is obviously impracticable here to attempt to present a detailed outline of future research, however, some of the broader problems can be touched upon. It can safely be stated that we shall never have an adequate basis for specifications, design and construction in concrete and reinforced concrete until we develop a sound theory. If we had a complete theory of concrete we should be able to calculate in advance all the properties and behavior of any concrete or reinforced concrete member or structure. At present practically every different mixture of concrete materials presents a new and unsolved problem.

Workability is the starting point of every phase of concrete work—yet the most fundamental phases of this subject are still undiscovered. What are the characteristics of the individual constituents of a concrete mix which dictate the amount of water which must be used in order to produce a mix of a given degree of workability? There must be well-defined characteristics of the materials which can readily be measured which will give us the answer to this problem. What influence has the specific gravity of the cement, and its fineness and composition? What are the characteristics of the aggregate, size and shape of particles, specific gravity, surface condition? What properties of the water (surface tension, viscosity, fluidity, specific gravity)? In our previous studies of workability we may have kept too close to the problem. Suppose we broaden the problem and make it a study of the factors governing the plastic state of any granular material mixed with any liquid. In this case concrete becomes only a special case of a much broader problem. I believe it is only along such lines that we will learn the true principles underlying workability of concrete.

There are many differences of opinion regarding the relative merits of different types of aggregates which would no doubt be reconciled by adequate research.

A similar analysis should be made of the elements of concrete mixing. What are we endeavoring to accomplish when we put a batch of concrete in a mixer and turn it over for one minute? No adequate theory of concrete mixing has been presented.

There is need of a better correlation between cement tests and the properties and behavior of the same cements in concrete.

There is much work to be done in the development of proper methods and mixtures for concrete floors.

The committee on Properties of Mass Concrete has pointed out many new problems, most of which have not yet been touched upon in previous investigations.

The studies made by our committee on Durability of Concrete have been made by the members of the committee and presented to the Institute as a voluntary contribution. It is doubtful if the entire annual budget of the Institute would have been sufficient if we had been called upon to pay the cost of this work.

The reports presented at this convention by the committee on Reinforced Concrete Column Investigation is an excellent example of the manner in which the Institute can sponsor research projects. This program of tests carried out at the University of Illinois and at Lehigh University will involve an expenditure of about $40,000.00, all of which has been contributed from sources outside the Institute. If the Institute were required to pay all the costs of such a program the cost would have been double the figure given. It was only due to the interest of the Universities and the loyalty of the men in charge to the Institute and its aims that this work could be carried at an expenditure which represents only the cost of labor and materials.

The Institute has no research laboratory, it has no research staff, it has no funds which are adequate to finance an extensive research program, yet it has at its disposal many research laboratories and in its membership most of the leading experts on concrete. Experience in the column tests has shown that adequate funds can be secured for a well-considered program of tests. If all these factors can be brought together, it will furnish

a method of adding immensely to our knowledge of concrete and concrete materials.

We hear a great deal about over-production; however, this condition is not yet apparent in the field of research.

Mr. Walter Lippmann, Editor of the New York *World*, a keen observer of modern life, has pointed out that the most remarkable development of the past 25 years is not the machine age or mass production, but rather that men have discovered how to make discoveries. In other words, research has been reduced to a formula. Any engineering problem can be solved if we have the will to solve it.

This view should encourage us in the belief that our problems can be solved if they are attacked with skill and courage.

While it is natural that I should emphasize the importance of research, I do not mean to minimize other phases of our work, such as specifications, design, construction and shop manufacture. However, research covers all of these departments and is, in fact, of no value until it is translated into office and field usage. Research is the handmaiden of design and construction. The writing of adequate specifications will become child's play when we have at hand adequate information upon which to base the various requirements; this information can be furnished only by research.

Studies of Concrete Mixtures*

BY S. C. HOLLISTER†

In the course of concrete mixing and placing it has been the experience of many that there are occurrences in the behavior of the mixture that are not readily explained by the customary concepts of mixtures. For example the following queries arise from experience in concrete mixing:

1. What causes the "fatness" that is acquired by concrete mixtures on prolonged mixing or working?
2. Why is there sometimes a crazing and checking of trowelled surfaces?
3. What causes shrinkage?
4. Why is there a change in consistency of mix on prolonged mixing?
5. Why is there frequently a lack of bond between coarse aggregate and matrix?
6. Is there any relation between heating during prolonged mixing and the physical state of the mix?
7. Why does prolonged rodding improve the strength of concrete?
8. Is uniformity of consistency a measure of the desirable objective of the mixing process?
9. What are the characteristics of cements that often affect the mixing, placing and handling of the concrete mixture?
10. Can water be withdrawn from mix after placing—how much can be withdrawn safely and what is the result?

These and other problems arising in the concrete craft are not in the writer's opinion explained by the customary concepts of the composition of a concrete mixture during its workable state.

It is the purpose of this paper to relate many of the phenomena mentioned above, through the medium of a more detailed concept of the concrete mixture prior to hardening.

Concrete in a structure consists of an arrangement of variously sized inert particles called aggregates bound together by hardened cement. At the time of casting, concrete consists of the aggregates, cement and water. The change from wet concrete to

*Presented at the 27th Annual Convention, A. C. I., Milwaukee, Feb. 24-26, 1931.

†Professor Structural Engineering and Assistant Director Materials Testing Laboratory Purdue University.

hardened concrete involves the chemical activity of the cement made possible by the water present in the mix. The quality of the finished concrete is dependent upon the amount of cement, the amount of water and the nature, size, amount and gradation of the various aggregates.

There are two major characteristics by which the concrete mixture is usually judged; (1) its placeability and (2) its strength. A concrete for a given purpose must first of all be of such a plasticity that it may be properly placed. After it is once placed it is then judged by its strength and other correlated qualities. In both these characteristics the amount of mixing water is of prime importance. Anyone with practical experience knows the relation of water in producing a workable mix. Abrams[1] has established the dependence of the resulting strength primarily upon the relationship of the amounts of water to cement. Talbot and Richart[2] developed in the mortar-voids method of proportioning the same relation in another form.[3]

In the practical use of concrete the workability of the mixture is of especial importance. It is not only dependent upon the amounts and forms of the dry ingredients, but also upon the amount of water used in mixing, and especially upon the consequences of wetting these ingredients when considered together with the circumstances of mixing and placing. Thus the study of the train of events attendant upon the addition of the water assumes increasing importance. Stress is laid upon the importance of considering the ingredients *as they occur in the mix*, and the effect upon such occurrences of the various circumstances common to the mixing operation.

WATER AND SAND

When a grain of sand is slightly moistened two things transpire: (1) a small portion of this water is absorbed into the body of the grain of sand; and (2) the unabsorbed water surrounds the grain of sand in a film, which film is held in position by the force of surface tension.

When two grains of sand are moistened and brought together they are prevented from making direct contact by the film of water. If the amount of water surrounding the two particles is

[1]References are listed at the end of this paper.—EDITOR.

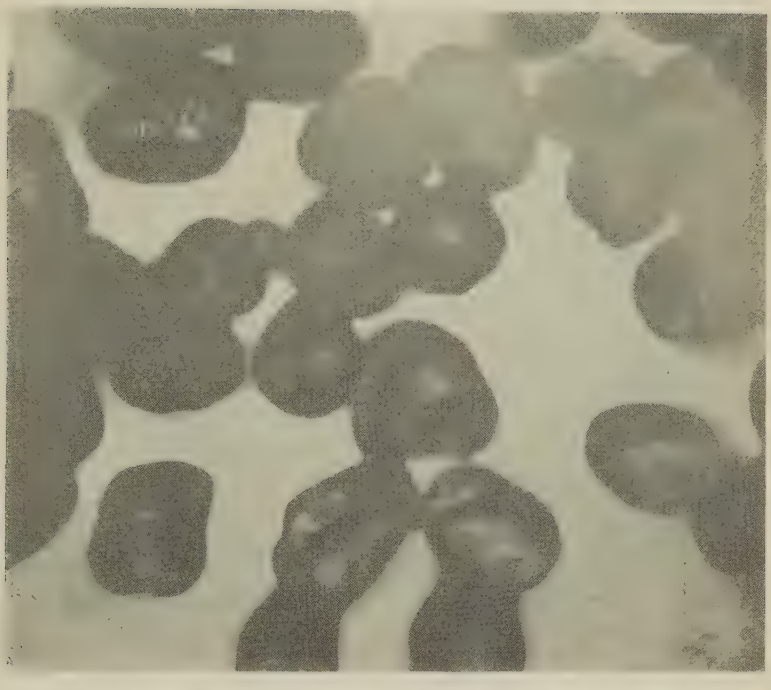


FIG. 1—LIQUID FILM ON GRAINS OF STANDARD OTTAWA SAND

small there is an insufficient amount of water to form an envelope to encompass both particles at once. Instead each grain is covered with an independent surface film which undertakes to maintain its separate form. The consequence is that the film of water holds the two particles from coming in close contact, because each grain behaves as though it were now larger than originally by the increase in diameter of the thickness of the water film. (See Fig. 1).

If the water surrounding these two particles is now gradually increased this excess water collects in the neck between the two grains and the water surface now tends to form a "peanut shape" surrounding both particles. It is now seen that a considerable amount of surface tension is acting to pull one particle toward the other. Under such conditions the two particles are no longer held apart.

The above discussion explains why dry sands are increased in bulk upon gradual moistening and why upon further increase of moistening a point of maximum bulking is reached beyond which further increase of moisture gradually diminishes the bulking until it almost entirely disappears.[4]

It can be demonstrated that for a given liquid the amount of moisture required to produce maximum bulking in a sand also the amount of the bulking are functions of the surface area of the sand.[5] A fine sand bulks more than a coarse sand and requires more liquid to produce this bulking because the surface area is greater for the fine sand.

A given sand will, of course, readily support the moisture required to produce maximum bulking without tendency of this moisture to drain off. This amount is not sufficient to produce a consistency in the wet sand comparable to that of the average concrete mixture.

WATER AND COARSE AGGREGATE

Assuming an impervious material, when gravel is moistened a film of water is formed on the surface of each particle in much the same manner as the film of water is formed around the grain of sand. For a given amount of gravel the surface area of the particles is less than for the surface area of a similar volume of sand. The amount of water necessary, therefore, to moisten the given volume of gravel (that is, to produce the film on the surface of the particles) is very much less than for the same volume of sand.

Whatever water is necessary to wet the surface of the gravel particles will not be sufficient to produce an appreciable mobility of the gravel. If, however, this wetted gravel be combined with a little more than enough of the wetted sand than would be required to fill the gravel interstices greater mobility will result. Even this amount of water, however, will not produce a workability such as is desired for concrete.

If the stone possesses much absorptive power at the time of wetting, a considerable amount of water will be taken up in saturating the stone. If the absorption is rapid, additional water must be supplied in order to give lubrication to the stone in the

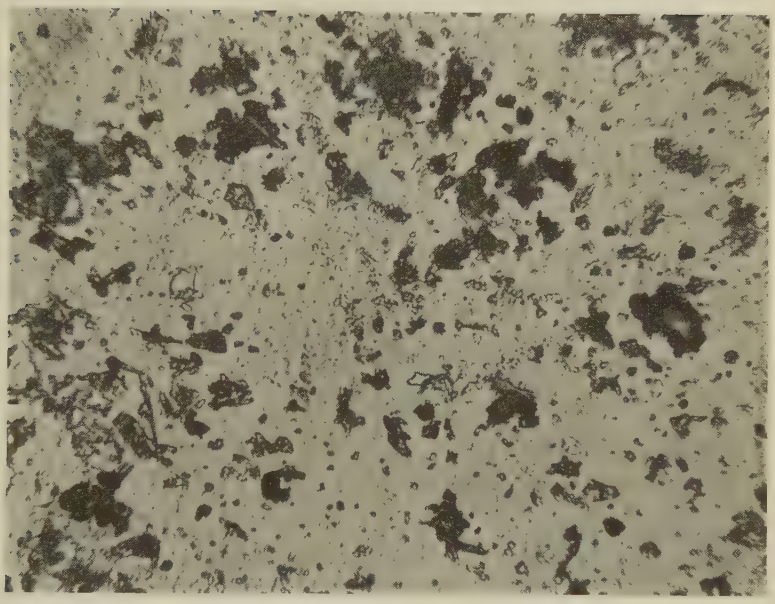


FIG. 2—DISPERSION OF CEMENT IN WATER (x275)

mix. On the other hand if the absorption is slow, less water will be needed by the stone to render it workable in the mix.

The shape of the stone particles not only affects the ease with which these particles move over one another in the mix but it also affects the amount of water required to wet the surface.

WATER AND THE CEMENT

The wetting of cement is a considerably different phenomenon from the wetting of sand. One need only think of the relative diameter of a grain of cement to the thickness of the water film to realize that a cement grain would be entirely inundated by such a film.

As shown in Fig. 2 the cement particles are of varying size and the fine particles tend to cling to the surface of the larger ones. When water is brought into contact with such a cluster of particles it engages the entire cluster, because the thickness of the surface film of water is relatively so large there is no tendency to separate

or disperse the particles and wrap each one individually with a film.

While in this instance an excess of water was present in order to obtain proper illumination, the tendency of cement to occur in clusters has been observed in a mass of more compacted structure. In either case as the water was quickly evaporated the voids between grains of cement are readily discernible.

THE MECHANICAL MIXTURE

Let us now bring together the three ingredients: the wetted gravel, the wetted sand and the wetted cement. Our first consideration is the mechanical arrangement of these wetted particles to produce a uniform dispersion. The sand grains fill the interstices between the particles of gravel, and in addition force the particles of gravel somewhat apart. The cement particles are to be found in clusters adhering to the particles of sand and gravel. The water exists in a film around the sand grains, the particles of gravel and the clusters of cement.

It will be found that where the water is just enough in amount to produce these films the mixture will be too stiff a consistency for proper working. Any additional water stirred into the mixture must occur in the small interstices remaining between the wetted particles of ingredients. This water may be termed "free water" in the mixture. Its only function is to aid in producing workability.

It should be noted that the water producing the film on the several ingredients is there for the sole purpose of producing workability for the mixture with the single exception of the water assigned to the cement. It is now important to consider the reaction between the cement and the water assigned to it and the effect upon the cement of all other water occurring in the mixture.

THE HYDRATING PROCESS

When water comes in contact with cement there is begun a long train of events, the conduct of which is the deciding factor in the extent to which this cement may impart strength to any given mixture. These events may be divided into two groups: those occurring prior to solidification and those occurring after solidification. It is not the purpose of this paper to go into

detail concerning the events in the second group. A large amount of research has already been done to determine the nature of such activity and the results of these researches have been published elsewhere.[6, 7, 8, 9] It is, however, desirable to develop here the processes occurring prior to hardening of the cement because it is while those processes are going on that the manipulation of the given mixture is possible. Whatever is accomplished prior to hardening will, of course, affect the processes subsequent to hardening, and conversely, what is left undone during the period in which manipulation is possible is not recoverable subsequent to hardening.

The question has often been raised whether a film of air is entrapped when the water first encompasses a cluster of cement grains or a single grain. The writer has been unable to observe under the microscope the presence of any such film. It is quite certain that the process of colloidal formation begins very soon after the water is added. If there is a film of air, therefore, separating the water and the cement it is very likely that this air is absorbed by the water. If this were not so and the air was present it would indeed be very difficult to initiate the colloidal formation.

If water is added to cement and is at once rapidly evaporated there is evidence of a cementious action having taken place. It would not appear that this could be so if there was an insulating film of air about the cement, unless this film of air were rapidly broken down. It is the writer's opinion that in either case the film of air, if at all present, is not of great consequence in our present consideration.

The water begins to dissolve the surface of the cement particles in contact with it. The substance thus produced in solution forms a jelly or colloid.[10] The strength of this solution depends upon the amount of water immediately at hand, which water causes in proportion to its amount a dilution of colloidal substance.

Observation of successive stages of colloidal formation and behavior develops many important facts which explain other phenomena noted in the process of mixing and handling the concrete mass. The amount of water usually present under ordinary conditions of mixing is greatly in excess of the amount of water

necessary to hydrate the cement. In addition to this water immediately in contact with the cement particles there is the large amount of water on the surface of the sand and coarse aggregate particles.

For some minutes after the water has come in contact with the cement no change is visible in this water. Gradually the water develops a variation in density so that it appears to wrinkle, giving much the appearance of wavy window glass. Then quite suddenly the waviness becomes very pronounced so that it looks like a shrunken film of varnish. At this point the formation of the colloid is quite well advanced. It is important to note that throughout this process of colloid formation the cement particle does not dissolve but remains as a nucleus in the colloidal mass.[13]

The action of the water in diluting the colloidal substance requires special attention. The more water present for a given amount of colloid the weaker that colloid. Furthermore the weaker the colloid the greater the amount of volumetric shrinkage upon the evaporation of the included water. Since the entire setting processes take place only in the colloidal substance present the weaker that colloid the less its physical strength. Fig. 3 shows clearly this phenomenon. The clusters of dark spots forming the boundary of the pattern in the figure are cement particles around which has been formed a rich colloid while these particles were submitted to hydration. Water extended over the entire area of this group and by this water colloidal substance distended, becoming the more dilute the farther it spread from the cement particles around the margin. Now, as the water gradually evaporated, all of the colloidal material shrank, the greatest shrinkage taking place in the central area of the pattern where the colloid was weakest. In this area the colloidal film thus formed was too weak to withstand the shrinkage stresses, hence producing cracks. It will be seen that these cracks are widest in the area of weakest solution and that they taper gradually and finally disappear as they approach the marginal area in which occurred the rich colloid.

It is thus seen that the water occurring in excess of that necessary to form a rich colloid becomes a rapidly weakening influence. This would appear to support the soundness of the physical law enunciated by Abrams that the strength of a port-

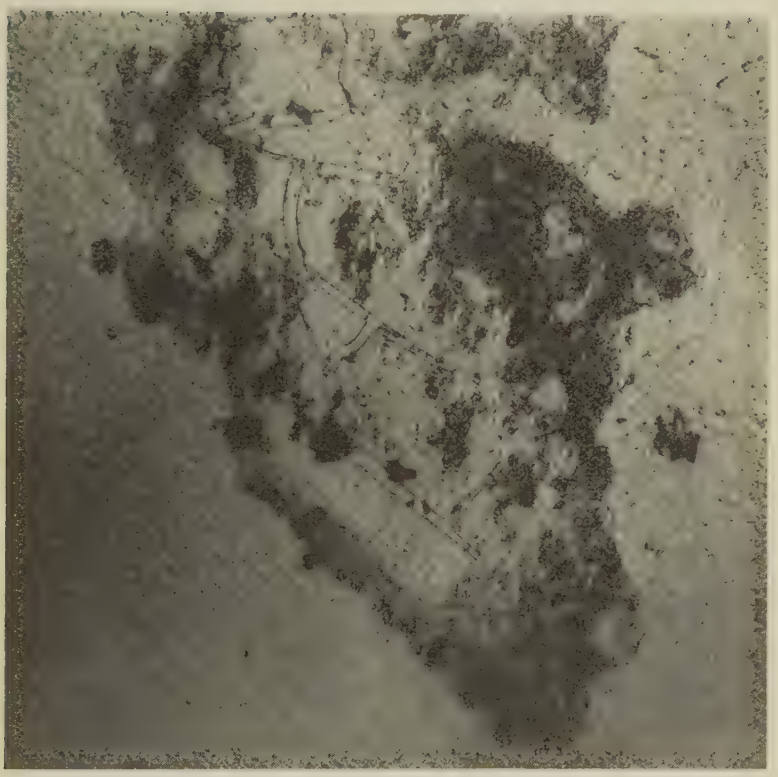


FIG. 3—CRACKING OF SHRUNKEN COLLOIDAL FILM DUE TO GROSS EXCESS OF WATER (x450)

land cement concrete mixture is a function of the ratio of the amount of water to the amount of cement in the mixture.[1]

While the above described phenomenon is taking place about the particles of the cement, other physical changes are taking place in a concrete mixture. From the time the colloidal "syrup" begins to form there is a gradual rise in the temperature of the mixture. This rise appears to continue until the stage of gel-formation, at which point the rate of temperature rise increases for some minutes. Thereafter the former rate of temperature rise continues up to the beginning of hardening. The consistency of the mixture during the period of colloid formation remains nearly

the same with a slight reduction in slump. At the time of gel-formation the consistency is noted markedly to stiffen. Thus a mixture with a 4-in. slump may stiffen to a 1-in. slump or less on formation of the gel.

With the use of a blend of two standard portland cements at a room temperature of 70 deg. F., evidence of "syrup" was noted from 10 to 15 minutes after the water was brought upon the cement. The gel state occurred at from 40 to 45 minutes after wetting the cement. These values are different for different brands of cement.

Whether the "syrup" or gel mentioned above represents the hydration of aluminates only, prior to hydration of the silicate group, or whether both are proceeding simultaneously or whether the state noted originates from another source than those just mentioned the writer cannot say. For the purpose of this paper the significant thing is that there is this jelly-like substance surrounding the cement particles and that its presence has a bearing upon the workability of the mix.

Another test was made to determine the volumetric effect on a block of neat cement of the shrinkage of colloidal substance noted in Fig. 3. A given amount was mixed with a gross excess of water and agitated from time to time for a period of six hours, after which it was allowed to settle and harden. It was noted that the upper portion of the mass of cement was "fat" and slimy, whereas the lower portion was very much less fat. Upon hardening in a smooth glass container it was found that a glaze formed next to the glass, and that this glaze, at three days, had developed a considerable adhesion to the glass. Upon air curing there appeared checks in the top face and these checks developed into cracks extending down halfway from the top face. (See Fig. 4.) The cracked portion was that portion formed of excessively wetted colloidal substance noted before setting took place.

It should be noted that the amount of water was far in excess of that in a concrete mix and that the amount of shrinkage is not at all comparable to that to be found in concrete unless placed under water. The significant relation is that between shrinkage and excessively wetted colloid, and that such colloid is readily floated to the top surface of a mix, when the mix is wet, or when

it is considerably agitated. Over-working of the top surface by troweling or otherwise will promote this accumulation of "fatty" mortar and if dried too rapidly will produce cracking. Many floors and sidewalks have exhibited this tendency.

Microscopic examination of the glazed surface above mentioned disclosed cracks in the glaze itself soon after exposure to the air.

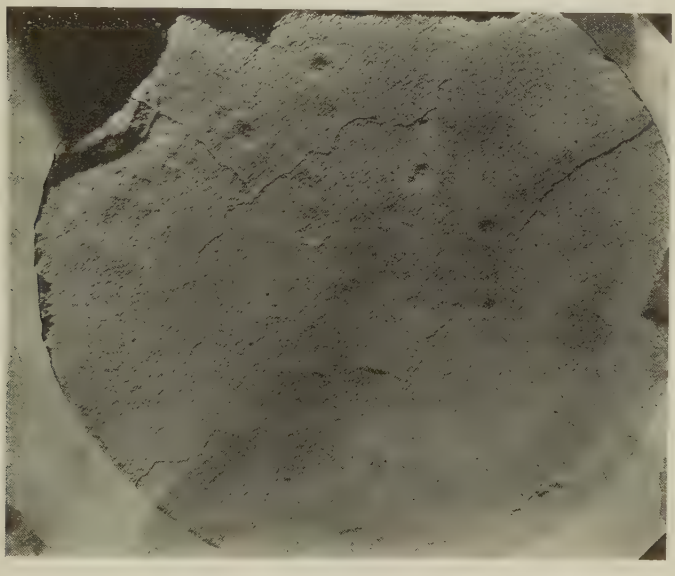


FIG. 4—CRACKS IN BLOCK OF NEAT CEMENT CAUSED BY GROSS EXCESS OF WATER

It would appear that there is a relation between crazing or surface checking and the shrinkage on loss of water in excessively extended colloidal substance collected at the surface. Technique which would tend to work the colloidal material *away from* the surface to be exposed would seem to reduce the occurrence of crazing.

Shrinkage on drying of a concrete mass, aside from temperature effects, would seem to be related to the richness and wetness of the mix, since these factors are the elements involved in extent of colloidal formation.

The foregoing test indicates one other important action.

Excess water does not produce more colloid of the desired richness, but instead dilutes (and hence weakens) such colloid.

The question now naturally arises, What is the relation between water and colloid? If the colloid goes into solution the escape of such solution would also mean the escape of what was originally colloidal substance, and there would consequently be a loss of strength proportionate to such escape.

W. I. Freel, of the Materials Testing Laboratory at Purdue University, made some tests to determine the solubility of cement in water. He found that the agitation of cement in an excess of water for various periods of time followed by settling of cement particles until the top liquor was clear, produced a top liquor which when decanted had a specific gravity of unity. This would indicate that the colloidal substance does not go into solution, but instead, is diluted but not dissolved by the water.

Tests were made to determine whether the liquor formed by the method used by Mr. Freel would show any sign of cementious characteristics when mixed with a known sand to form a mortar. No cementing ability was found by such tests.

FACTORS ARISING FROM CHARACTER OF AGGREGATE

There appear to be various views among investigators as to the proportion of strength of concrete contributed by the mortar and by the coarse aggregate. The coarse particles cannot contribute to the strength of concrete unless a bond between them and the mortar can be established.

As previously described a film of water is present on the surface of the coarse aggregate at the beginning of the mixing operation. Particles of cement lodged against this film cannot readily penetrate it. And for the number of such particles there is too much water in the film to admit of a satisfactory degree of colloidal strength being formed in the film that will reach through the film to the surface of the aggregate.

If this film of water cannot escape until after final setting a poor attachment to the stone is effected by the mortar. That such poor attachment is often obtained in concrete is shown by the fact that on fracture numerous examples occur in which the coarse aggregate itself is not fractured but instead pulls away from the matrix.



FIG. 5—WATER VOIDS AGAINST BOTTOM OF COARSE AGGREGATE IN OVER-WET MIXTURE

A microscopic examination was made of numerous specimens of which on the one hand the aggregate had pulled away and on the other the aggregate had fractured. In general it was found that where the aggregate pulled away a slight glaze was noticeable on microscopic examination, this glaze being similar to that formed by cement colloid alone upon evaporation of the moisture. Examination of the socket in the matrix out of which the aggregate was removed showed a surface consisting of very fine particles of sand together with the cement, the whole surface displaying a pitted sugary structure.

It is exceedingly difficult to obtain a satisfactory photograph conveying a correct impression of the observation made on the plane of contact between mortar and certain types of stone.

Fig. 5, however, is a piece of over wetted concrete on which some of the faults of excessive water just described are plainly visible. The concrete specimen was photographed in its upright position as cast. At the top are plainly seen portions of two sockets from which the stone fell away from the matrix. Arrows point to voids existing along the underface of a fractured stone. Voids such as these are often found in mixtures less wet than the one shown in the figure, especially if the aggregate has low absorption at the time of mixing.

A comparison of the properties of aggregates before and after use in concrete indicates clearly that for the particular types of stone observed, the bond to that stone was effected by sufficient available absorption on the part of the stone to pull in the water forming the film. Fracture of such absorbent stone was effected if the matrix was as strong as, or stronger than, the stone itself.

The effect of the absorption of water by the aggregates in affecting the strength of the concrete was found to result in an increase in the strength of such concrete commensurate with the change in the water cement ratio.

Increase in strength made possible by lowering the water cement ratio through the use of absorbent aggregate gives rise to speculation as to possible improvement in the quality of concrete produced by removal of excess water by this or other means. The investigations of Pearson[11] at the Bureau of Standards upon portland cement stuccos indicates the importance of the removal of water from the fresh coat by the proper amount of absorption by the undercoat. The same principle is involved in the use of sand molds in cast stone work. The undesirable results of excessive absorption by untreated gypsum molds indicates the necessity of control of such absorptive processes. The technique developed in the studio of John J. Earley[12] involves, in part, removal of excess mixing water through controlled capillary absorption.

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CONVENTION DISCUSSION

W. A. Slater—Mr. Hollister has spoken of the development of something like a jell around the cement, as one of the agents in producing workability and that the effect is greater after continued mixing. I raise no question as to that. I do not know anything about it, I have to admit, but I want to call attention to a development within the last few weeks at Lehigh University. We were testing a conveyor designed to transport concrete from the central mixing plant to the place where it is to be used. It is a closed drum conveyor. A mix was designed using seven gallons of water per bag of cement. We charged the conveyor and at various intervals from the time of leaving the laboratory, for two and a half hours, samples were taken from the drum and analyses made of them, separating the ingredients into their various constituents and measuring them. We found that the amount of cement apparently increased until, after two hours and a half, from our measurement we would say that we had about thirty to forty per cent more cement than when we started out. We were not being fooled entirely, because the amount of water remained constant, the amount of sand decreased and the amount of decrease in the sand was just about the same as the amount of increase in the cement. We concluded that we were grinding up the sand as we went along, but the point I wanted to call attention to particularly was that the slump regularly decreased from the beginning until the end, although the amount of water remained constant. We concluded that the amount of material ground up in the operation had an important bearing on the amount of slump. It is entirely possible there were other factors at work too, but I think that must have had something to do

with it. Having nothing to do with this particular part of it, but as a matter of interest, the strength also increased as the time from the beginning of the transportation increased. I do not remember how much but it was a very substantial increase for the specimens taken after two and a half hours over those taken at the beginning.

P. H. BATES—I am particularly pleased to see that Professor Hollister is injecting a little physics into his engineering course because I think if his students once sense that this whole matter of making concrete is chemistry, and chemistry a branch of physics, we will have gotten around to the fundamental basis on which to study concrete.

Professor Slater cites the interesting discovery that in mixing concrete he made cement. Nearly 20 years ago a well-known contractor in Pittsburgh had a contract for grouting some veins in a mine shaft in order to prevent leakage of the water into the shaft. After some time he noticed that the bottom of the shaft in which he was not supposed to do any grouting was filling up, and he had to stop his grouting He measured the amount of grout he had in the bottom of his shaft and found that it was many times the amount of cement he had purchased, and he was very much alarmed about the whole matter.

I think we have got to realize that this matter of the setting of cement is a colloidal phenomenon, nothing more nor less, and will have all the characteristics of a colloidal material. You can assume it is an inorganic glue (probably you are sitting on a chair fastened with an organic glue) and you know exactly what happens when you do not treat an organic glue properly; if you use too much water or not enough, it does not glue. It is the same way with cement, and runs through many, many similar phenomena. A great many of you have noticed the work done by Miller at Minneapolis, the effect of alkali on concrete. If he cures concrete at a high temperature and puts it in alkali, it is almost immune to attack. You can dilute the white of an egg with water indefinitely, but if you once subject it to a high temperature, it is no longer soluble. Professor Miller probably has done that in treating his concrete at a high temperature.

For further discussion which may develop, see JOURNAL *for Oct., 1931 (Vol. 28.) Such discussion should reach the Secretary by Sept. 1, 1931.*

The Use of Color in Concrete*

Progress Report of Committee 408—Recommended Practice in the use of Color in Concrete

W. D. M. ALLAN, CHAIRMAN

INTRODUCTION

The American Concrete Institute appointed Committee 408 in July, 1930, to draft a Recommended Practice for the Use of Color in Concrete because of the growing demand for information on this subject.

The principal classes of color materials are: 1. Pigment admixtures; 2. Selected aggregates; 3. Chemical stains; and 4. Water and oil vehicle paints.

The Committee planned to make each of these sections the basis of a separate report or to suggest to the Institute that committees be appointed to cover each one.

Of the four classes of color work, there is most urgent demand for information on the use of pigment admixtures in troweled concrete surfaces which is the basis of the Committee's first report. Based on a study of the whole problem of the use of color in concrete, the Committee suggests the following subjects which might be developed as additional separate reports:

Pigment admixtures

1. In monolithic concrete walls
2. In cast stone and precast units.
3. In stucco and mortar

Selected aggregates

1. Color possibilities of natural aggregates.
2. Color possibilities of manufactured aggregates.
3. Combination of colored aggregates and colored mortars.
4. Surface treatments of concrete to develop color value of aggregates and mortar.

*Presented at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.

Chemical stains

1. Chemical composition of stains and reaction with concrete to produce color.
2. Application of chemical stains to concrete surfaces.

Paints

1. Composition of water and oil vehicle paints.
2. Application of paints to concrete surfaces.
3. Protective value of paints on concrete srufaces.

Much has been written in technical papers on these subjects and a great deal that is of value has been presented to the American Concrete Institute which could be assembled under proper headings, supplemented by more recent information and put into usable form.

In presenting this recommended practice for the use of pigment admixtures in troweled concrete surfaces, the Committee has followed each section where it seemed desirable to do so with a sub-section of discussion. This method makes possible the inclusion of explanatory material for those who desire it, without making unnecessary reading for those who are interested only in the specific recommendations.

The Committee hopes that a large number of members, users and manufacturers interested in color, will comment freely on all parts of the following proposed recommended practice to the end that all who desire to get thoroughly permanent colors in their troweled concrete work may do so.

There are several points in this report upon which there is important division of opinion, e. g., several members feel that the use of the dust coat is sufficient to produce satisfactory work; while others feel equally certain that the dust coat is to be avoided on all but unimportant work. It must be admitted that many satisfactory dust coat jobs have been produced and in the hands of skilled workmen with close supervision, this method has possibilities. On the other hand, it must be admitted that excess of fines on the surface of concrete wear or weather surfaces leads to dusting and crazing. No satisfactory basis has yet been developed for evaluating these two ideas or arriving at the proper method of handling. From the anticipated discussion in the JOURNAL and from suggestions and criticisms received direct by committee members, it is hoped to be able to present for

adoption, a Recommended Practice for tentative adoption at the 1932 convention.

Progress Report on a Proposed Recommended Practice for the Use of Pigment Admixtures in Troweled Concrete Surfaces

PURPOSE

This recommended practice is designed to furnish the best information available on the selection and handling of pigment admixtures, laboratory tests for their quality and methods of using pigments in troweled concrete surfaces.

PART 1

General Requirements for Pigments

Only inorganic pigments, as limited in this section, should be used in troweled concrete surfaces. Synthetic and natural mineral pigments are satisfactory if they meet the following requirements:

1. Insoluble in water.
2. Free from soluble salts.
3. Free from acids.
4. Fast to intense sunlight.
5. Fast to heat up to 300 deg. F.
6. Not to exceed 2 per cent by weight of free moisture.
7. Fast to alkali and weak acids.
8. Free from calcium sulphate in excess of 15 per cent by weight.
9. At least 90 per cent of synthetic pigments pass a 325-mesh sieve and at least 90 per cent of natural pigments pass a 200-mesh sieve.

Discussion—The manufacturers of concrete pigment admixtures should be in a position to guarantee that materials meet these requirements. The test procedure which appears as appendix to this report will assist the purchaser in getting a prompt report from a recognized testing laboratory covering the suitability of a pigment based on the requirements above.

Synthetic pigments usually have a high degree of purity and uniformity. They are more brilliant and the cost per pound is usually higher than for natural pigments. For particularly important jobs and where brilliant colors are desired, synthetic pigments are recommended. For dull colors, natural pigments meeting the requirements above will be satisfactory, provided the amount of pigment added does not reduce the quality of the concrete so that it will not resist wear and weather. Frequently the smaller amount of syn-

thetic pigment at a higher price per pound will produce the results desired at a lower cost than the natural pigments.

In general, the finer the pigment the greater its coloring value.

Calcium sulphate sometimes occurs as a filler in pigments and it is necessary to limit the amount because of the detrimental effect of the sulphate on the concrete.

Some pigments have the property of absorbing moisture from the air and care should be taken to keep them dry; otherwise, they will be lumpy. When they are mixed with cement and stand for some time before being used, the moisture may react with the cement to cause caking. The limit of 2 per cent of moisture is intended to apply to the free moisture in the pigment and not to chemically combined water.

Selection of Pigments

Red—Use either synthetic or natural oxides of iron.

Yellow—Use synthetic yellow oxide of iron.

Buff—Use either natural or synthetic yellow oxides of iron.

Brown—Use synthetic or natural brown oxides of iron.

Black, Gray—Use synthetic black iron oxide.

Green—Use only synthetic chromium oxide; Cr_2O_3, at least 98 per cent pure.

Blue—Use synthetic cobalt blue at least 98 per cent pure and free from sulphates.

Methods of Test

Wherever iron oxide pigments are required, the purity shall be at least 98 per cent iron in the form of Fe_2O_3 after ignition at from 900 to 1000 deg. Centigrade for synthetic pigments and for natural pigments, the purity shall be at least 55 per cent, determined in the same manner.

Pigments shall be tested for purity in accordance with the test procedure in the appendix to this report.

The amount of pigment shall not exceed the proportions of 12 pounds of pigment to one sack of cement, (94 pounds net).

Discussion—In general, the least pigment used to obtain the desired color value, the better. This is especially true of surfaces that are to be subjected to heavy wear or to water or frost action. When necessary to use the maximum amounts of pigment shown above, special care must be taken to place the colored concrete with a minimum of water, to increase the time of mixing and to get more than average curing.

The recommendation for the use of synthetic black oxide of iron is based on the difficulty of getting definite non-fading black color with other blacks frequently used. While some organic blacks have and maintain a deep black

color they are light and fluffy and difficult to mix. If sufficient time and care are taken to obtain a thorough mix they are satisfactory. Not more than 3 per cent by weight of cement of organic pigment should be used. Cobalt blue is recommended in preference to ultramarine blue because of the large amount of fading which in cases has resulted in a total disappearance of the color when the ultramarine blue has been used. For specific jobs where some fading is not objectionable and where the manufacturer will guarantee the durability of the ultramarine blue, within reasonable limits of fading, it may be used under the manufacturer's direction.

Pigments used as admixtures in concrete must be considered as fine aggregate. Experience shows that because of the fine grading of the pigment, which is desirable for color value, it dries up the mix rapidly, leading to the tendency to use more mixing water. This, in turn, reduces the cementing value of the cement paste and results in inferior concrete. As the amount of pigment is increased, it is desirable to decrease the amount of fines below the No. 50 mesh screen in the aggregate used. Care must be taken not to increase the amount of mixing water used per sack of cement, above that required to produce concrete of the quality needed for the job if it were not be colored.

Color Value of Pigments

No table of definite color values can be given for pigments in concrete. It is necessary for the user to prepare sample panels of convenient size and combine the pigment in various accurately weighed amounts with a definitely known amount of cement, aggregates and water, such as he will use on the job. All proportions must be accurately made and the whole mass thoroughly mixed, placed and finished exactly in the way it will be done on the job. The panels should then be cured in the same manner that the job will be cured and then allowed to dry. The surface of the panel should then be wiped off with a mixture of one part paraffin oil and one part of benzine or similar vehicle and allowed to dry. From these sample panels, the color values possible with different materials and combinations can be determined and then, by reference to the data covering the method of producing the original panels, the color can be duplicated within very narrow limits.

Discussion—Color values cannot be established except by trial because the fineness of the pigment, color of the cement and aggregate, quality and quantity of the mixing water, time of mix, amount of troweling and method of curing all affect the values obtained.

A record should be kept of the exact proportions used in each panel so that the results can be duplicated. If the source of any of the materials is changed, a new series of panels should be prepared. A slight variation in either pigment,

cement, aggregate, water or method of mixing, placing or curing will produce variations in the finished job.

Some members of the committee have strongly urged that some standard of color values should be developed. A standard color chart has been suggested and all colors could be graded as 100 plus or minus depending on their tinting value.

Another suggestion strongly advocated is that the color values be taken from color charts prepared by the manufacturers and the concrete prepared and placed according to the directions of the manufacturers. The committee does not wish to rule against either of these plans but it recognizes that neither of them is feasible at present and the problem raised is important and needs solution.

Blending Pigments

Considerable skill and adequate equipment are required to blend successfully two or more pigments. Contractors and other users of pigments with a good "eye for color" can enlarge the range of colors available by blending or mixing the primary colors. This is frequently necessary on polychrome jobs and where slight variations in colors are desired. It is recommended that blended colors be purchased from reputable color manufacturers unless the user is sufficiently experienced and equipped to blend his own pigments. An accurate record of the exact amounts of colors used, weighed to the nearest ounce, for one-sack batches is absolutely necessary to get satisfactory results with blended colors.

Proportioning Pigments

All the materials entering into the batch must be carefully weighed. The quantities of pigment and other ingredients to be used are determined from the sample panels. Uniform results require that every batch be proportioned, mixed, placed, finished and cured exactly like every other. Even though the aggregate, cement and water may be measured by volume, the pigment must be measured by weight.

Discussion—Measuring pigments with a dipper or by any other volume method may be satisfactory for rough work where uniformity and high grade wearing surfaces are not required but this method is unreliable even with skilled workmen for important jobs. If the pigment has to be mixed on the job the color at least should be weighed out accurately at the plant or shop and delivered to the job in containers, each one containing just enough pigment for each batch.

Mixing Pigments

Pigments must be thoroughly mixed with the cement and with the cement and aggregate to get uniform work. There are several ways in which the pigment and cement can be mixed and the methods are described in the order of their value:

The color mixer, which is a tight, revolving metal drum with stationary paddles or blades, or a stationary drum with revolving paddles or blades of sufficient capacity to hold one-half, one or more sacks of cement with the pigment, is the best method for mixing pigments. The cement and pigment are accurately weighed and placed in the mixer and mixed for not less than ten minutes. When the batch is mixed it is sacked and kept dry until it is used.

In the absence of a color mixer on important work, it is recommended that factory-prepared colored cement be purchased from a reputable manufacturer.

Another method that will prove satisfactory if care is used, is to mix the accurately weighed proportions of cement and pigment by passing the two materials through a $\frac{1}{8}$-inch screen several times until all particles of pigment are thoroughly incorporated with the cement. This should be done at the plant where the materials can be kept dry and where weighing equipment is available.

Another method of mixing pigments on less important work is to use a concrete mixer. The aggregate and cement are put into the mixer and turned over a few times, then the measured amount of pigment is added in small portions, care being taken to get the pigment well distributed throughout the batch. This should be added in small quantities to avoid the danger of lumps forming which are very difficult to get thoroughly mixed. The mixing should continue until the pigment is uniformly distributed throughout the mass and then the water is added. The wet mixing should continue until the mass is of uniform consistency and the pigment is thoroughly distributed.

Discussion—Attempting to mix by hoeing together pigment and cement in a mortar box can produce satisfactory results only in the rarest cases and this practice should not be considered if uniform, reliable results are desired.

Regardless of the mechanical means employed, both the dry and wet mixing must be continued until the pigment and cement, or the pigment, cement

and aggregate are mixed to a uniform color. The best method of determining if the mixing has progressed long enough is to take a small amount of the material, possibly a shovelful and place it in a pan, wheelbarrow, or on a smooth concrete surface. Then spread it out in a thin layer with a trowel and trowel this thin layer hard. If streaks of pigment or cement can be seen, the mixing must continue until later trials show no streaking.

PART 2

Types of Troweled Colored Concrete Surfaces

In general, the use of color in troweled concrete surfaces can be divided into three approved types of construction:

Two-course monolithic construction, in which the colored top course is placed before the base course has hardened, so that the two courses harden as one-course construction.

Regular two-course construction, in which the colored top course is placed on a hardened base course.

One-course construction, wherein the pigment is used throughout the thickness of the section.

Discussion—Two-course monolithic construction is best practice wherever possible, because danger of a separation of the two courses is eliminated.

Regular two-course construction is used where it is not desirable to place the finish until some time after the base course has been placed.

One-course work is not common except in very thin slabs and in other work where only light tints are wanted.

The terminology developed for the three types of color jobs has been criticized as not definite enough. However, the changes suggested have been no improvement. Further study is needed.

Thickness of Colored Topping

For severe wear and exposure, the colored topping should be not less than 1 in. thick. For moderate wear and exposure the colored topping should be not less than ¾-in. thick. These apply to two-course monolithic work. For regular two-course work, all colored toppings shall be at least 1 in. thick.

Selection of Aggregates

Aggregates should be selected for grading, cleanness, durability and color.

For all severe wear and/or exposure, the aggregates should be composed of fine and coarse particles graded as follows:

Fine aggregate—All passing a ¼-in. screen, not more than 10 per cent passing a 50-mesh screen and not more than 3 per cent passing a 100-mesh screen.

Coarse aggregate—All passing a $\frac{3}{8}$-in. screen and all retained on a $\frac{1}{8}$-in. screen.

Aggregates should be hard, durable sand, pebbles or crushed materials free from dust, loam, clay or vegetable matter and contain no flat, elongated or flaky particles.

For moderate wear and/or exposure coarse aggregate may be omitted.

Aggregates should be selected from a source where the grading and moisture content remain uniform throughout the job.

Discussion—The grading of the fine and coarse aggregates should be such that there is a predominance of the larger sizes.

Where possible, the aggregates should be selected to match or strikingly contrast with the color of the mortar. When the mortar wears from the surface, the color of the exposed aggregate will have a marked effect on the color and appearance of the surface. Translucent aggregates, such as light colored, crushed marble, help maintain the desired color if the mortar surface wears away.

If the supply of aggregate is not uniform as to grading and moisture content, the user should screen his own materials and store them so that they are uniform.

Proportions of Cement and Aggregate

For severe wear and/or exposure, the proportions should be approximately one sack of cement, one cu. ft. of fine aggregate and two cu. ft. of coarse aggregate.

For light wear and/or exposure, the proportions should be approximately one sack of cement and two cu. ft. of fine aggregate.

The proportions of aggregate may be varied to correct for workability and consistency.

Discussion—The above proportions may have to be varied depending on the type of work, the grading of the aggregate and the amount of pigment used. In varying the proportions, as much of the coarse aggregate as possible should be worked into the mix. These proportions are on a damp and loose volume basis.

Mixing Water

Mixing water should be free from injurious amounts of acids, alkali, oil or vegetable matter and should be fit to drink.

The total amount of mixing water to be added with each sack of cement should not exceed 5 gal., including the free moisture in the aggregate. In adjusting the proportions of aggregate, correction for free moisture should be made.

Discussion—Impure mixing water, especially if it contains acids or alkali, may lead to efflorescence.

For particularly severe wear and/or exposure the total amount of water should be reduced to 4½ gal. per sack of cement. Accurate moisture determination for the aggregate must be made. When changing proportions of fine to coarse aggregate or of aggregate to cement, correction must be made in the amount of mixing water so that at no time the total amount of water added per one-sack of cement exceeds 4½ or 5 gal.

Consistency

Colored concrete for troweled surfaces should be placed as stiff as possible. The slump in no case should exceed 4 to 5 ins. Very little, if any water should appear on the surface when it is screeded or straight-edged.

Discussion—Stiff consistency is necessary to prevent the fine material coming to the surface during placing and troweling. The concrete should be stiff enough so that it has to be rolled or lightly tamped into place.

Mixing

Colored concrete, especially for two-course monolithic and regular two-course work, should be mixed not less than two minutes after all the materials, including the water, are placed in the mixer if the cement and pigment have been thoroughly mixed before being placed in the mixer. At least five minutes longer will be required for mixing if the pigment has not previously been mixed with the cement.

Discussion—The grading and proportions of aggregate recommended above, will tend to be more harsh than the average cement finisher is in the habit of working. Increasing the mixing time up to five minutes will improve the workability and at the same time increase the intensity of the color and improve the strength and wearing properties of the hardened concrete. The time of mixing for all batches should be the same.

Placing

Two-course monolithic and regular two-course work should be placed on bases that are clean and free from water. In *two-course monolithic* work, the base should stand until all the surface water has either evaporated or has been broomed or screeded off or removed in some other manner. All laitance or scum shall be removed before the colored concrete is placed.

In *regular two-course* work, the base should be broomed or cleaned and roughened just before it hardens. Before the colored concrete is placed, the surface should be scrubbed with water to remove all dirt. If there is oil or other foreign material on the

base, it should be removed with soap and hot water or with sharp edged tools. When the washed surface has begun to dry slightly, a mixture of cement and water or one part of cement and one part of sand mixed to a thick creamy consistency should be broomed onto the surface in a uniform layer about $\frac{1}{8}$-in. thick. Before this grout coat begins to dry the topping should be placed.

The colored concrete should be deposited in small batches close together and screeded or straight-edged to grade. Special care shall be taken when straight-edging to fill up and prevent low places in the topping. The surface should then be wood floated and allowed to stiffen before the steel trowel is used. Areas of colored topping should be placed, small enough so that by the time of finishing all of the topping will have stiffened to the same degree.

Discussion—Any water on the base, whether it be from the fresh concrete or applied for washing, should be removed or allowed to evaporate before the colored concrete is placed. Otherwise this water will rise through the colored concrete, tending to cause efflorescence and dusting. In two-course monolithic construction the danger of water from the base rising through the colored topping will be reduced to a minimum by placing the base as stiff as possible.

Troweling

Steel troweling should be delayed as long as possible and still be able to finish to a smooth compact surface. Troweling should be reduced to a minimum and the final strokes be made all in one direction. Sprinkling the surface with water to aid in troweling should not be permitted.

Discussion—Any working of the surface even with a straight edge or screed tends to bring fines to the surface. Steel troweling is particularly bad. The surface should be worked as little as possible. Fines lead to dusting, checking and blotchy color. If the troweling is delayed until all the water sheen has disappeared and the concrete has stiffened slightly, the danger of drawing fine material to the surface is reduced to a minimum. Attempting to fill up low places by troweling surrounding material into them will result in discoloration and the tendency toward dusting. Low places should be leveled with the straight-edge and wood float.

Filling Low or Wet Areas

Filling in low or wet places with dry mixtures of pigment and cement or pigment, cement and aggregate should not be attempted. Steel troweling wet places should be delayed until the excess water has disappeared. Low places should be filled with a concrete mixture exactly the same as is being placed and care

must be taken not to work the concrete in the low places any more than any of the other surface. When filling a low place, the concrete for several inches around should be roughened and then brought back to grade along with the added concrete by using the straight-edge and float.

Curing

All colored concrete surfaces should be damp cured for not less than seven days. Where subjected to severe wear or exposure the damp curing should continue for not less than 10 days. The surface at no time during these periods should show any tendency toward drying. Ponding, completely covering with damp sand or with approved grades of heavy waterproofed paper or fabric should be required.

Discussion—Nowhere in concrete work is proper curing of more importance than in troweled colored concrete surfaces. Strength, resistance to wear, freedom from dusting and checking are as much dependent on proper curing as upon any one factor. In hot, dry or windy weather, the surface should be protected from the sun and wind immediately after steel troweling. The curing should be started as soon as the surface has hardened enough to prevent damage.

Surface Treatment

After the concrete has been cured as required above, it should be washed off with clean water and allowed to become thoroughly dry. If after washing a light colored deposit or film, called efflorescence or bloom appears on the surface or if the color seems to have faded, the surface may be dampened with water and washed with a solution of one part of muriatic acid to 10 parts of water. The acid is put on uniformly over the surface with a long handled brush or broom and allowed to remain for not more than five minutes, when it is entirely removed with an abundance of clean water. It may be necessary to repeat the acid treatment. The acid should not be allowed to remain long enough to start etching the surface and should not be used at all on heavy wear surfaces.

Efflorescence may be removed by steel brushing or rubbing with steel wool. The material loosened in this way should be broomed from the surface before it is washed or given any other surface treatment.

Rubbing the surface with a mixture of equal parts of paraffin oil and benzine or naphtha will remove the efflorescence if it is not too thick. This treatment is sometimes used to brighten the colored surface.

Washing the surface with liquid soap which has been allowed to stand overnight and then scrubbed off with warm water improves the colors and wearing surface. This treatment should be repeated two or three times over a period of four months.

Ordinary floor wax aids in brightening the colors and filling the surface so that it is more easily kept clean.

There are numerous other commercial waxes and resins which applied to the surface will aid in brightening colors.

This progress report has been submitted to the following committee, which voted as follows:

W. D. M. Allan, Chairman
C. A. Cooke
J. G. Ahlers
M. R. Rust
J. W. Ayers
E. M. Waldo
John Mott
Rexford Newcomb

Of whom 7 have voted affirmatively, one negatively and none has refrained from voting.

Respectfully submitted,
W. D. M. Allan
Chairman, Committee 408.

Appendix—Report Committee 408

Laboratory test procedure for determining the quality of pigment admixtures

Acidity of Pure Oxides

Boil five grams of pigment with 200 cc. of distilled water in a 300 cc. Erlenmeyer flask for 20 minutes. Cool under the tap and filter through a 15 cm. filter paper into another similar flask. Wash with about 40 cc. water. Add three or four drops 1 per cent phenolphthalein indicator, and titrate with $\frac{N}{100}$ sodium hydroxide.

Standardization of Sodium Hydroxide

Weigh accurately a convenient quantity (suggest .06 grams) of acid potassium phthalate $C_8H_5O_4K$ (obtained from the Bureau of Standards). Dissolve in cold water and titrate with the $\frac{N}{100}$ sodium hydroxide, using phenolphthalein as before.

The normality of solution equals $\frac{\text{weight } C_8H_5O_4K}{\text{burette reading x } 0.204}$

Percent H_2SO_4 in sample equals $\frac{\text{burette reading x } .049 \text{ x normality x } 100}{5}$

Blank tests should be run on the water used both in standardizing and the actual determination.

Routine Test for Presence of Acid

A ten gram sample of pigment is allowed to digest in 150 cc. of distilled water one minute. Filter off the pigment and analyze the filtrate with methyl orange or phenolphthalein.

Test for Moisture

Weigh out accurately a one to three gram sample of pigment into a fused quartz crucible, which has previously been thoroughly cleaned, heated to a temperature of 125 deg. C. cooled in a desiccator, and weighed.

Place the crucible, with the material, in the electric furnace, and heat to 110 deg. C. for one and one-half hours. Remove from the furnace, place in desiccator until cool and weigh.

Computation: Loss of weight divided by the original weight times one hundred equals percentage of moisture.

Determination of Percentage of Ferric Oxide

Solution: Dissolve ½ gram oxide in about 20 cc. 1-1 hydrochloric acid in 100 cc. beaker. Add enough stannous chloride to keep the solution very nearly in the reduced state. (If solution is over-reduced, add enough permanganate solution to restore yellow ferric color.)

Reduction: When the solution is complete, reduce the ferric oxide by adding stannous chloride drop by drop until the solution is entirely colorless. Then add one drop in excess. Wash solution into a 400 cc. beaker containing 250 cc. of water, add 20 cc. titrating mixture and 10 cc. of mercuric chloride. Titrate with potassium permanganate to a permanent pink color.

Computation: Percentage of Fe_2O_3 = burette reading x normality factor x 200.

Potassium Permanganate Solution: About 4 grams per liter or of such strength as to give 1 cc. equal to 1/100th gram ferric oxide.

Stannous Chloride: 100 grams stannous chloride, 100 cc. concentrated hydrochloric acid, 100 cc. water. Make up to liter and add a few pieces of metallic tin.

Titrating Solution: 160 grams manganese sulphate, 1320 cc. sulphuric acid, 1.84 Sp. Gr., 330 cc. of syrupy phosphoric acid, 1750 cc. water.

Mercuric Chloride: Saturated.

Determination of Sulphur (gravimetric method)

Mix thoroughly 1 gram of the finely ground pigment with 8 grams of sodium carbonate and from 0.5 to 0.7 gram of sodium nitrate, in a large platinum

crucible. Heat carefully over a Bunsen burner or blast-lamp until the mass appears perfectly liquid and in a state of quiet fusion. Run the fused mass well up on the sides of the crucible and plunge the crucible half way into cold water. When cold, treat in a tall narrow beaker with boiling water until all soluble salts are dissolved, and no hard lumps remain. If the solution is colored blue or green by $Na_2Mn_2O_8$, add a few drops of alcohol. The solution will now be colorless, or yellow if chromium is present. Allow the undissolved matter to settle and decant the solution through a filter into a small beaker. Wash the residue with hot water once by decantation and then pour it upon the filter and wash four or five times. Add hydrochloric acid to the filtrate until just acid, evaporate to dryness and heat one hour longer at 110 deg. C. Moisten the residue with hydrochloric acid and set on the hot plate for a few minutes, add 50 cc. water, heat till all soluble salts are dissolved, filter and wash until the filtrate measures 85 or 90 cc. (If the silica in the ore is low, evaporation to dryness is unnecessary. Simply acidify and filter.) Heat this latter to boiling and add 10 cc. of a 10 per cent solution of barium chloride, previously diluted to 30 cc. and brought to boiling point. Allow to stand for some hours, filter, wash with hot water, ignite and weigh as barium sulphate. This weight multiplied by 0.13732 and the product divided by the weight of the sample gives the per cent of sulphur in the pigment.

Specific Gravity

Employ a 50 cc. pycnometer. Insert an accurately weighed portion of sample, fill half full with water, boil gently to remove any air, cool under the tap to room temperature. Fill to the mark with fresh distilled water; after noting the temperature, weigh. Call this weight *C*. Empty flask, clean and refill with distilled water to the mark. The temperature of this test should be identical to the first procedure. Again weigh and call this weight *P*. Let *W* equal weight of dry sample.

Computation: Sp. Gr. $= \dfrac{W}{P + W - C}$

Note: More finely divided pigments which have a tendency to form a film on the water seem to work better when some other liquid, lighter than water is used. Kerosene gives excellent results.)

Computation: Sp. Gr. $= \dfrac{W \times G}{P + W - C}$ where *G* equals the Sp. Gr. of the liquid used.

Determination of Chromium Oxide[1]

Solutions: Sodium thiosulphate—tenth normal—24.82 grams per liter ($Na_2S_2O_3 . 5\ H_8O$).

Potassium hydrogen sulphate—136 grams per liter ($KHSO_4$) (approx. normal)

Sulphuric acid—one part acid—one part water.

Starch solution—Make 2 grams soluble starch into a thin paste with cold water. Pour into 200 cc. boiling water.

[1]See Scott's Standard Methods of Analysis.

Standard permanganate—about 4 grams per liter. Same as used for iron. *Determination:* Heat gently 0.3 gram sample with about 3 grams sodium peroxide (Na_2O_2) in a covered porcelain or Vitreosil crucible.[2] The crucible furnace should be just below redness. Heat for 15 minutes after fusion is liquid.

Cool and place crucible in 400 cc. beaker. Dissolve melt in about 100 cc. cold water, adding some hot water to hasten solution. Boil 10 minutes. If $Fe(OH)_3$ separates out in appreciable quantity it must be filtered off. See Scott's Standard Methods of Analysis. Make nearly acid (litmus) with sulphuric acid. (The solution darkens.) Then add 15-20 cc. $KHSO_4$ sol. and boil for 10 minutes longer.

Cool under the tap. Make up to mark in a 250 cc. volumetric flask. Pipette 50 cc. aliquot into a 300 cc. Erlenmeyer flask. Add about 25 cc. sulphuric acid. Dilute to about 200 cc. with cold water. Add about 2 grams potassium Iodide (KI) and after a few minutes titrate with the thiosulphate solution. When the green color begins to predominate, add a few cc. starch solution and finish titration drop by drop until the blue color fades to pale green.

$$\text{Then per cent } Cr_2O_3 = \frac{\text{cc. } Na_2S_2O_3 \text{ x value 1 cc. x 500}}{0.3}$$

Standardization with Standard Permanganate

Pipette 20 cc. $KMnO_4$ into 300 cc. Erlenmeyer flask. Acidify with 20 cc. H_2SO_4. Dilute. Add about 2 grams potassium iodide and, after a few minutes, titrate with standard thiosulphate, arriving at end-point exactly as described in method.

Reactions: (a) $2\ CrO_3 + 6\ KI = Cr_2O_3 + 3K_2O + 6I$.
(b) $I_2 + 2Na_2S_2O_3 = 2\ NaI + Na_2S_4O_6$.

Calculations:

$$\text{Normality of permanganate sol.} = \frac{\text{wt. Fe equiv. to 1 cc. } KMnO_4}{0.056}$$

$$\text{Normality of thiosulphate sol.} = \frac{\text{cc. } KMnO_4 \text{ x 20 x normality } KMnO_4}{\text{cc. } Na_2S_2O_3}$$

$Fe_2O_3 \times 0.6994 = Fe$; then 1 cc. N. $Na_2S_2O_3 . 5\ H_2O$ should
$= .056$ Gr. Fe $= .01733$ grams Cr.
therefore: 1 cc. thiosulphate sol. $= 0.0253$ grams Cr_2O_3 x norm.

Determination of Cobalt

The cobalt is determined after the removal of the lead manganese (iron and zinc) by precipitating with ammonia and ammonium sulphide as cobaltous sulphide; dissolving this precipitate in nitric acid, adding H_2SO_4 taking down nearly to dryness, then diluting with water, making almost neutral with ammonia, and precipitating with a boiling hot concentrated solution of ammonium hydrogen phosphate.

The precipitate is washed, dried and weighed as $CoNH_4PO_4$.

[2]Weight of sample may be varied.

DISCUSSION

BY M. R. RUST*

Introduction—Dust coats are productive of unsatisfactory results and are only a method for the cheap installation of color with resultant inferior wearing surfaces. There are no happy mediums—a job, on completion, is either good or else no good. Dust coats must definitely result in the latter. It is practically impossible to obtain a floor, dust coated, which will be uniform in either color or the mix of the dusting materials. Furthermore, it is well-nigh impossible to find a floor, prior to dusting, which would have a uniform degree of moisture to hydrate the cement uniformly in the dust mix. Even in the hands of skilled workmen, dust coats will be found satisfactory on only a small portion of their work.

General requirements for pigments—Synthetic and natural pigments are satisfactory, according to the heading, if they meet the qualifications enumerated—and the 4th qualification set forth is "fast to intense sunlight." I have yet to find my first natural color which will meet this requirement, in fact, I might add that I have yet to find one which will closely approach such performance.

It seems a hardship on the contractor to determine whether he is adding only enough natural color to make sure that he does not produce a finish subject to wear. On the same subject, who desires a dull colored concrete surface? The object of using color is to avoid dull monotony and obtain rich decorative beauty. I have encountered comparatively few requests for dull colors and as a manufacturer, we eliminate any thought of such a result from all literature and recommendations. Even the layman appreciates that if it is possible permanently to color concrete to rich intense tones, it is logically possible to produce a dull effect. To criticise the endorsement of natural colors for dull color effects, further, I wish to point out that if a job were completed with the desired dull effect, using a natural color, it would within a few weeks fade to a most unsatisfactory effect.†

*Member Committee 408.

†Without presuming to "take sides" as between synthetic and natural pigment admixtures, may the Editor interpose as a mere suggestion that dull (*un*brilliant) tones are not infrequently desirable in garden landscapes where concrete may be called upon to play a part in naturalistic effects in which brilliant tones would "scream" at nature.—EDITOR

Selection of pigments—I definitely oppose the endorsement of natural colors wherever listed under this heading. As regards the last color, blue, I must confess myself straddling the fence for the good reason that I cannot deny the more stable qualities of Cobalt compared with Ultra-marine—but on the other hand, I fail to see why Ultra-marine is not the more logical choice. Cobalt is unquestionably expensive—almost prohibitive in its pure form whereas Ultra-marines, of the finest preparation, are reasonable and for all practical purposes, will answer most satisfactorily. Despite opinion to the contrary, our experience has been without fault in the sale of such a blue. Admittedly, there are Ultra-marines of most questionable quality and some determination might be made between desirable and undesirable Ultra-marines. It seems that again we come to the same question for debate and that is whether to endorse something for the general good of the industry or to make our recommendations, in the final analysis, for the select few. My suggestion is to favor the multitude.

Methods of test—Nine pounds of pure synthetic color are ample to produce a rich and intensely colored concrete surface. Additional is only waste. Unquestionably nine pounds is more favorable to strong concrete than twelve and the only excuse for permitting the use of twelve pounds is to allow for the use of natural colors—on which I have already commented.

I am confident that our best recommendaton on the use of black is to stand entirely behind synthetic black oxide of iron. I have, personally, conducted a good many experiments to produce a black with more intensity but invariably something is lost with each gain. Not only is the mixing of the color with the concrete extremely difficult but other blacks or blends seem to produce a retarding action which would be highly objectionable and disconcerting to the unknowing contractor.

If we are to assume it true that the color present in a mix tends to dry up the mix and require more water, my suggestion of not more than nine pounds of color per sack of cement is preferable.

Blending pigments—To endorse the contractor's blending of colors is to invite more worry and grief than I care for. The contractor with a huge organization might be interested in

blending—but it is doubtful if he would care to spend the time when he can purchase the exact tone, required, from the manufacturer. Furthermore, presume that he is later required to build some addition to the original job—who possesses the formula for the blend? Here again, we are leaving ourselves wide open. It will probably be best to leave the business of blending to the maker of color with the facilities and knowledge required in blending.

Mixing pigments—Factory prepared colored cement is an excellent product in theory but it has many limitations. For the contractor to use colored cement, he must be prepared either to pay a considerably increased cost for the colored portion of the job or to anticipate less efficiency and color value in his completed work than he would receive through the purchase of the color and intelligent use on the job. Someone must pay the traffic for the machining of the color and the cement, not to mention a few other expenses involved.

Thickness of colored topping—I am again straddling the fence on this subject. I find that the bulk of the work is done with ½ in. topping and while I feel that added thickness is undoubtedly an advantage, we might wisely consider that with each ¼ in. added to thickness of the topping, the color cost is increased by a sizable sum. We may follow a rather common practice of endorsing a definite thickness with the expectation that we are to receive at least 50 per cent of that on completion—but on the other hand, contractors of the Jerry Builder type are hardly likely to follow even that reduction. If we specify ¾ in. and they are accustomed to laying ¼ in.—I strongly believe they will continue to lay just ¼ in. My thought is this, that if we can safely specify the use of ½-in. topping for, say two course monolithic work and ¾ in. for set slab construction, we might do better to adopt such thicknesses.

I am strongly inclined to believe that in attempting to recomment the sand, we are perhaps only adding confusion for the contractor. If nine pounds of synthetic color are used per sack of cement, and the sand is of a grading which will produce good, strong concrete, the finished color will be satisfactory and the slight variation of local sands would produce but very little

difference in the result. Simplicity, to me, is highly desirable in connection with endorsements to be made, widespread, to the contracting fraternity and I am confident that elimination of all trifles or incidentals will aid in getting what we, after all, desire most and that is something bordering on intelligent use of color in concrete.

Surface treatments—My experience with muriatic acid as a cleansing agent has been most unsatisfactory and I should, at no time, care to recommend it other than as an extreme measure. I am most doubtful of it being used successfully by a contractor without etching the finish and such a finish is hardly an attractive one to examine. We do not feel too kindly toward the use of steel wool as it tends to dull the finish but we do strongly favor the finish being rubbed with the mixture of about one-half paraffin oil and one-half naphtha. I agree that we should include after-treatments but I feel that acids should be recommended with restrictions.

I am entirely confident that whatever form our recommendations take in their final stages, we should make every effort to eliminate all trivial matters pertaining to the use of color and seek maximum efficiency in simplicity. I am confident that the bulk of the color used in concrete will be used by the smaller contractor and for this reason, I feel it wise to direct our efforts toward him. For every large colored job, there will be countless small ones and the small jobs will greatly govern the continued patronage of the public in colored concrete.

It would seem that much of the success of colored concrete must revert to some extent to the manufacturer of the color. If he, selfishly, sells an inferior color for the sole purpose of making a sale, he is, indirectly, destroying his own market.

Publishing standard methods of testing or determining colors will make an interesting file for a very limited few but just where shall we find the contractor who will utilize such information? Would it not be better for the Institute to adopt standards and issue these standards to the various color makers with the request that they brand their product, viz.: "This color conforms fully with the standards of the American Concrete Institute for purity and composition?" The manufacturer who would wilfully

misbrand a product would leave himself open to redress on the part of the public.

Uniformity seems a desirable feature in connection with color. We adopted the first step toward simplicity, almost three years ago, by recommending the use of nine pounds of our color per sack of cement—regardless of which of the 12 standards we manufacture. We later pioneered another step by packing all colors in sacks, each containing nine pounds, with the sole recommendation to use one sack of color for each sack of cement used in the topping. Other manufacturers might wisely follow this policy to the ultimate benefit of the industry and more particularly to the benefit of the smaller contractor—who, for some time to come, appears to be our largest and most important factor to consider. He is the man who will make or break colored concrete and while he may be partly lacking in appreciation of architecture, he is our one important objective.

CONVENTION DISCUSSION

Austin Crabbs—Mr. Allan dwelt on the importance of getting just the right amount of color in each batch. It might be interesting, in that connection, to know that one color manufacturer is prepared this year, on request, to furnish color in individual packages having the right amount of color to use in one sack of cement.

Mr. Allan—That indicates an excellent tendency; several manufacturers are doing that now. But suppose the company sends a package to you and a like package to somebody in the northern part of Iowa, and both of you use it; you will get different colors. You take the one pound package of color and use it with your local cement and develop colors that are practically the same within very, very narrow limits, from time to time, but they will be different colors from those in other parts of the state.

Mr. Crabbs—Perhaps I did not make myself quite clear. One color may be seven pounds to the package, another 10 pounds—furnished this way only upon request. He will not furnish those individual packages unless you tell him what your proportions are, how much cement you are going to use and how much fine

aggregate and how much coarse aggregate, and I think that is helping the contractor considerably.

CHAIRMAN COLLINS—In other words, it would enable you or the other manufacturer that Mr. Allan spoke about in the northern part of the state, always to have the same color, although his might differ from yours.

JOHN W. LOWELL—We have given a good deal of study to color in concrete, and one of the most baffling problems is the machinery for mixing cement and color. I was going to ask Mr. Allan if, in their investigations, they have developed any information along that line?

MR. ALLAN—Nothing beyond what you already know, recommending that the color be mixed for a minimum of ten minutes in a regular color mixer, not a ball mill, and we suggest that a small amount of that material be troweled to see if there are any streaks under a steel trowel. That would determine how much more than ten minutes is required. I think a definite effort is made not to reduce the fineness of the material, that is not to grind it as we have sometimes done in the ball mill, but thoroughly mix it in small paddle mills.

NORMAN M. STINEMAN—I believe the members would be interested in a series of articles published in *Concrete* in August, 1928, May, June, November and December, 1930 and January of this year. While these articles cover the manufacture of colors, the author, E. T. Ellis, gives at the end of each article the formula for different shades. For instance, these six articles cover yellows, reds, blacks, greens, blues and browns, and at the end of each article he gives the formula in the form of so many pounds of cement and so many pounds of different colors for a light shade of that particular color, or for a medium or a dark shade.

A. L. ASHER—In Mr. Allan's report there are two very important points—first and most important of all is the classification of mineral pigments. There are few people who know anything about the classification of mineral pigments. I feel that the manufacturer of each pigment should state on his container exactly what the material is, whether it is a chemically pure oxide, a

commercial oxide, reduced color, or natural pigment. That is one thing that I think should be absolutely standardized. The next point is the matter of mixing, inasmuch as the only mineral pigments we can use are those which are insoluble in water. The problem is to get a uniform distribution of color throughout the mortar. I think that the logical thing is to have the color ground with the cement. Even with a ball mill, I feel that the additional fineness you get there will be of no disadvantage at all, and you do get an almost perfect distribution of color throughout the cement, which means uniformity in the cement paste. When the color is brought on the job it must be measured. You cannot measure a material uniformly that runs 90 to 98 per cent through a 325 mesh. If the contractor knew that four bags of a colored cement would lay a hundred square feet of floor one inch thick, it would eliminate a lot of his troubles in figuring his costs and doing the work on the job.

H. F. Hedderick—Mr. Allan recommended a blade type color mixer. Several months ago we were interested in trying to find a color mixer and the Portland Cement Association recommended the pebble mill type. We were able to find those without much trouble, but their cost is extremely high. We tried to find a machine of the blade type, made especially for mixing colors. The only one we could find took half a bag to a bag of cement at the time, not large enough for our work. I wonder if the machines Mr. Allan recommends are on the market in the size to take five or six bags?

Mr. Allan—For such quantities you would have to use a regular stucco mixer. We were figuring on one sack batches and I think the regular stucco mixers would be best for the large batches you mention.

Nolan Browne—I should like to call attention to the use, in color work, of mineral pigments with an integral accelerator and hardener. We find that the advantage is threefold; first, their use accelerates the mixture and necessitates that the mechanic get off the job a lot quicker, thereby getting more work done; secondly, a thinner coat will wear the same length of time as a thicker coat, due to the hardening effect; and third, it reduces efflorescence on account of the waterproofing quality of

the admixture. There is a small amount of efflorescence when the material is first cured because the water does bring a certain amount of salt to the surface during the setting-up process; but when once cleaned, it has been our experience that efflorescence does not reappear; whereas, without a waterproofer, dampness is present and efflorescence may recur time and time again. I think it would be a good thing for the committee to give some consideration to this. In our own experience we have reduced the slump on the colored product to one and a half inches. Now that does make a very hard mixture to handle unless the mechanics are quite familiar with it, but it surely does get the results.

A MEMBER—I'd like to ask a question about blue color. Several years ago we used an ultramarine blue and applied it as a face on roofing tile. We used about ten pounds of color to a bag of cement; cured it in a steam room for about a week and put it on the roof and in about three months the blue color had faded entirely into a gray white. We took that roof off and made a mix of about ten pounds of blue color to the bag of cement and made a creamy mixture and blew it on the tile with a gun, about $\frac{1}{16}$ to $\frac{1}{8}$ in. thick. We put that tile back on the roof, and the roofing tile is a rich blue color. I would like to know why the color failed in the first instance and why it stood up when blown on with the gun.

MR. LOWELL—Are you sure that the tile color had not faded by the time you took it out of the curing room?

A MEMBER—No, it did not fade until we got it on the roof.

MR. LOWELL—I know that blue will fade under sufficiently high temperature.

ROY L. PECK—I'd like to ask Mr. Allan if he has any recommendation to make as to the use of color in surfaces of outdoor swimming pools to eliminate the use of color throughout the wall?

MR. ALLAN—We have limited this report to troweled surfaces.

MR. ASHER—The gentleman asked a question as to fading of ultramarine blue. There are two definite ultramarine blues. One is ultramarine blue that is absolutely a reliable blue and made for the concrete industry. The other ultramarine blue is

made for the paint industry. My conclusion is he was using a commercial ultramarine blue and got an excessive action of alkali. Using it with the gun after the concrete had hardened, he did not get that excessive action of alkali.

Benjamin Wilk—It seems to me that much would depend on the fact that, mixing it integrally the particles were not coated as thoroughly as in the thick surface application. In our experience, with a lean mixture and about six pounds of color to the sack of cement, we found that the trouble was in the color washing off the surface of the aggregate. There was not enough thickness of the color.

Mr. Asher—Well, the color in both cases was applied to the facing; we used the same amount of color in each instance.

Readers are referred to the Journal *for October, 1931 (Vol. 28) for further discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1931.*

Proposed Recommended Practice for the Manufacture of Concrete Building Block and Tile

*Report of Committee 708**

P. M. WOODWORTH, AUTHOR-CHAIRMAN

FOREWORD

These recommendations are intended to aid manufacturers of concrete building block and tile to make products that meet standard specifications of the American Concrete Institute[1] and to produce them economically. In addition to recent developments in the manufacturing procedure and in the use of porous aggregates, considerable attention is given to plant layout. This recommended practice is intended particularly for plants with an average daily production of 500 or more units. Much of the material will be of value for smaller plants.

A. PLANT LAYOUT

1. General Considerations

The economical production of concrete masonry units starts with the design of the plant which should be based on the probable volume of sales that will be developed during the succeeding three to four years. The plant layout should be such that new producing units can be added as the volume of sales makes it imperative without rearranging or disrupting the initial layout. Most manufacturers of concrete products machinery have plant layouts which are well adapted to their equipment. However, a layout fitted to one particular set of machines may not be entirely

*This report, by Mr. Woodworth, Author-Chairman of the committee has been submitted for preliminary advice and suggestion to the members of the committee: E. W. Dienhart, C. J. Herzog, M. I. McCarty, Eugene F. Olsen, W. P. Stanton and Stanton Walker and presented at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931, for discussion. It is published with convention discussion (which was all too brief) for further consideration and is open for general discussion scheduled for publication in the JOURNAL for Oct. 1931 (Vol. 28)—available to the Secretary by Sept. 1, 1931.

[1]American Concrete Institute Standard Specification P-1-A-29 is an appendix to this report.

suitable for another, consequently the type of equipment should be selected before the plant layout is definitely settled. There are a number of general considerations regarding plant layout which are common to most types of equipment.

Discussion—In establishing a concrete building block or tile plant a carefully prepared market analysis is the first step. The total volume of sales that will be developed during the next three to four years will determine the size of the plant that should be built. Many manufacturers have definitely precluded any chance of economical manufacture by building plants that are too large and over capitalized. Unwarranted optimism regarding volume of business seems to be the rule in starting a concrete building units plant. An important percentage of the high-fixed-capital plants established during the past five years have not been profitable. This should not be interpreted as meaning that small, poorly equipped plants are profitable but that to be profitable the size of the plant should be in proportion to the business that will be developed with an aggressive sales organization.

2. Elevation, Grading and Drainage

Raw materials and finished products should be handled by gravity wherever possible. The elevation of the plant floor should be sufficient to provide drainage away from the plant in all directions and a slope of about one-eighth in. per foot away from the machine to the end of the runways.

Discussion—In general the block or tile machine is at the highest point on the plant floor to provide sufficient slope from the machine through the curing rooms and out to the far end of the yard whether industrial cars, lift truck or gravity conveyors are used. The yard should be graded slightly below the concrete runways or the level of the industrial tracks because the yard tends to fill up with loose pieces of concrete, sand, gravel, etc. causing pebbles to collect on the runways, making operation of cars or trucks difficult.

In laying out a products plant, grade stakes for the yard, curing room and main factory floor should be carefully set first. Frequently a few loads of material used to build up the elevation to the proper grade in the beginning will save a manufacturer a great deal of trouble and expense a year or so after the plant is in operation.

3. Conveying Equipment

The use of conveying equipment other than that ordinarily provided with the machine is advisable if the labor saved more than offsets the interest on and depreciation of the additional equipment. Whether or not lift trucks, industrial cars, elevators, belt or roller conveyors, batch cars or other conveying equipment should be installed depends on the amount of reduction in the cost of manufacture obtained with this equipment.

4. Arrangement of Equipment

The more compact the arrangement of the plant and still provide for freedom of movement of men and equipment, the lower the labor cost. Aggregate storage and cement supply should be placed as near mixers as possible.

Discussion—The tendency to limit the amount of space at the side and in front of machines so that lift trucks or cars cannot be manipulated easily to and from the machines should be avoided. While it is important to save all possible extra steps, these should not be saved at the expense of cramping the movement of material through the plant.

Provision for ample skylights and artificial illumination is important. Painting the inside of the plant a light color is desirable.

5. Cement Handling

Cement storage facilities should be provided to keep the cement dry whether it is handled in bulk or in sacks. Bulk cement, due to the tendency to "fluff," should be measured by weight.

6. Aggregate Proportioning Equipment

Accurate equipment for proportioning aggregate either by weight or by volume is essential. Such devices should work quickly and easily. Overhead bins with the other necessary equipment for handling will usually prove economical for plants regularly producing 500 or more units daily.

Discussion—Several efficient weight batching devices are on the market and are to be recommended over volume measurements. No corrections have to be made for bulking of materials due to moisture. The only correction necessary is for the variations in moisture content in different shipments of aggregate.

7. Curing Room Construction

After units are molded, the quicker they are placed in favorable curing conditions the better the quality of the concrete. Curing rooms must be relatively tight to prevent air currents and loss of heat. They should have insulated roofs. The rooms are usually made wide enough to accommodate 2 racks, allowing 6 inches between radiation and the rack on each side and 10 to 12 inches in the middle. Tight enclosures should be used at both ends of the tunnel. The enclosures may be either moving doors made of well painted wood or light weight metal. Tight fitting canvas curtains are sometimes used on the plant end of

the rooms. Usually the doors are hinged at the top or slide in vertical guides which are made tight to prevent loss of heat and moisture. Curing rooms should be painted white on the inside to aid visibility.

Discussion—The tendency has been to build curing rooms too high. A temperature of 125 to 135 deg. F. is common against the roof, while at the floor it is not more than 70 to 75 deg. F. The heat should be kept down as low as possible and building the room no higher than is absolutely necessary to clear the top row of block is desirable. This will require workmen to stoop slightly while moving products into and out of the rooms but is no hardship.

If the layout of the plant makes it necessary to build the tunnels longer than can be filled in a half day's run, dividing curtains, usually of canvas, should be provided in the tunnel so that a run of 3 to 4 hours can be enclosed and curing started. This arrangement requires dividing the curing equipment so that each section can be regulated independently. In determining the length of curing tunnels, a minimum of 6 in. should be allowed between the ends of racks. The space between the radiation and the racks and between the racks in the center is to facilitate the movement of racks and the circulation of heat and moisture in the rooms.

8. *Types of Curing Systems*

There are six common methods used to provide satisfactory curing conditions for the freshly molded units. Refer to Figs. 1, 1A and 2.

Type "A" provides radiation and steam moisture control independent of each other. A header is placed within 6 in. of the

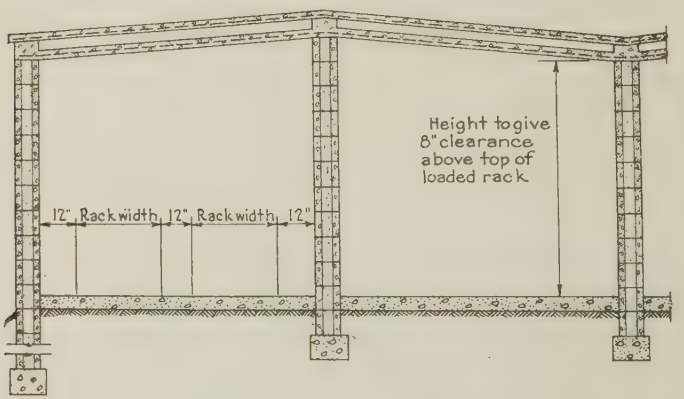


FIG. 1—TYPICAL LOW PRESSURE STEAM CURING ROOM

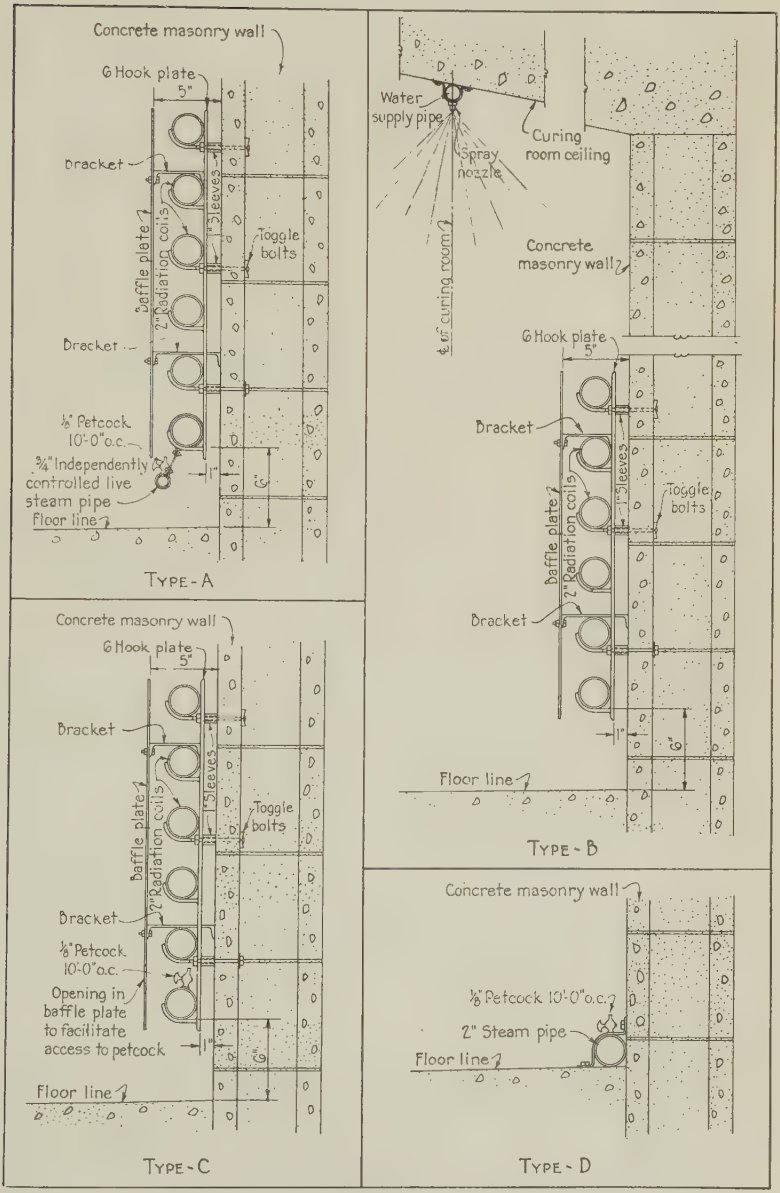


FIG. 1A—PRINCIPAL TYPES OF LOW PRESSURE STEAM CURING INSTALLATIONS

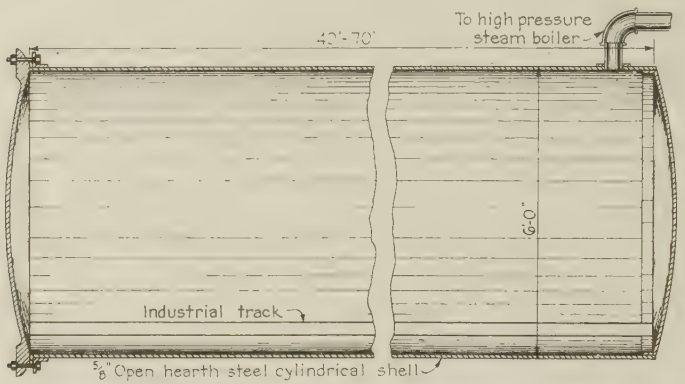


FIG. 2—HIGH PRESSURE STEAM CURING CYLINDER

floor at both ends and on both sides of the curing tunnel. Two-in. pipe connects these headers. Usually six pipes are required on each side in the northern sections of the United States if masonry walls and insulated roofs are used and if the rooms are of 2-rack width. The headers should be provided with three or four extra plugs so that additional radiation can be tapped in if the temperatures obtained are not high enough. The moisture is provided by steam through a ¾-in. pipe running just below the bottom pipe of the radiation system. This pipe is fitted with ⅛-in. petcocks on 10 foot centers staggered with those on the opposite wall. The petcocks should be directed upwards and at a slight angle toward the wall so that the steam is forced up through the radiation coils. Radiation and moisture are controlled by separate valves outside of the room. Cement-asbestos or galvanized iron baffle sheets are suspended 2 in. in front of the radiation coils. The bottom of the baffle sheet is placed even with the bottom pipe of the radiation. This acts as a jacket and causes a circulation of heat and moisture in the tunnel and prevents block directly opposite the pipes from becoming dry.

Type "B" provides independent steam coil radiation with moisture supplied through fog nozzles attached to the ceiling of the room. These fog nozzles are operated at water pressures of 30 pounds or more per square inch. If connected with the

regular water system, a heating coil should be provided so that the fog is of a temperature of at least 100 deg. F. when it enters the room. Baffle plates should be used in front of the radiation coil the same as is described for Type "A."

Type "C" provides radiation and steam moisture combined. The construction is the same as in Type "A," except that the bottom return pipe on the radiation coil is fitted with ⅛-in. petcocks on 10-ft. centers staggered with those on the opposite wall.

Type "D" provides for heat and moisture by exhausting live steam into the room through a single pipe placed within 6 in. of the floor on both sides of the room. This pipe is either drilled with ⅛-in. holes or fitted with ⅛-in. petcocks.

Type "E" provides for the curing of products in cylinders where the pressures vary from 100 to 125 lb. per sq. in. Special equipment is required similar to that used in the manufacture of sand-lime brick. While this method is not common at present, it seems to have good possibilities.

Type "F" is the least that can be expected in the way of curing equipment. It provides for rooms or sheds that are reasonably tight to prevent excessive air currents. They are equipped with fog nozzles to provide moisture. Usually this system is used where temperatures do not fall below freezing.

Discussion—Of the first four methods described, Type "A" seems to be best, because it enables the manufacturer to control independently the heat and moisture. In freezing weather or where it is necessary to get units on the job in a relatively dry condition, the mositure can be shut off after a period of moist curing and the products dried considerably by radiation. Type "A" also has the advantage of giving more thorough circulation of heat and moisture, thus tending to even the temperature and moisture conditions in the top and bottom of the curing room.

Type "B" is nearly as satisfactory as Type "A" but is slightly deficient in obtaining circulation in the curing room.

Type "C" provides ample radiation and moisture content. The drawback to this method is that the heat and moisture are controlled together. With this system it is nearly impossible to provide just heat, which is necessary sometimes for drying units after the moist curing.

Type "D," while commonly used, is not nearly so satisfactory because in order to get sufficient heat in the tunnel it is necessary to exhaust too large an amount of steam into it. Such an excess nearly always results in moisture

getting into the manufacturing part of the plant, causing unhealthy working conditions and deterioration of equipment, buildings and cement.

Type "E" should be thoroughly investigated by products manufacturers designing new plants because present indications are that the units can be cured in from 8 to 12 hours and have a strength at least equivalent to the 28-day strength under normal curing conditions. In addition, the units seem to be quite thoroughly dry and ready for immediate delivery when they come from the pressure cylinders. This makes for prompt delivery and reduces stock piles.

9. Summary of Plant Layout

Discussion—Fig. 3 shows a conventional layout embodying many of the above suggestions. In this plan the overhead bin (1) may be filled by any type of elevating equipment. The advice of the machinery manufacturer will be valuable in deciding on the type of elevating equipment to be used. The size of the bin is determined by the capacity of the plant and the regularity with which the material can be obtained. Generally not less than a three days' supply should be provided for. Opening (2) should be fitted with a measuring device, preferably weighing equipment for proportioning materials. A chute, weighing lorry or dump car (4) may be used for carrying aggregate to mixer.

The stock of cement should be piled on platform (3) close to mixer. Water tank (5) should be placed above mixer and out of the way of charging equipment. It should be provided with an automatic valve and water gauge for filling and with a perforated pipe for discharging. Space (6) under platform may be used for storing pallets, extra equipment, also a portion may be used for boiler room.

Mixer (7) on the raised platform discharges directly into agitating feeder from which concrete is elevated from machine hopper by conveyor (8). It is generally more satisfactory to have one mixer for each machine. (9)

A modification of the layout at this point is in common use. The overhead storage is elevated sufficiently so that the measuring equipment and mixer can be placed on a platform high enough to discharge directly into the hopper of the machine. Which of these two methods is better depends on the cost under local conditions. The cost of providing the extra height of storage space and elevating equipment should be measured against the cost of the agitating feeder and machine elevator. There is also a loss of floor space and more equipment to be cleaned in the first arrangement.

Concrete floor and yard runways slope from front of machine to far end of storage yard, usually ⅛ of an inch to the foot. Lift trucks (10), industrial cars or conveyors carry units to the curing room. Inside doorway (11) should be closed with vertically hinged or sliding doors or canvas curtains. Moisture in curing room is supplied as described under Section 8, "Types of Curing Systems" through coil (12) or by spray nozzles (13). Allow at least 6 in. between ends of racks (14). Outside doorway (15) should be closed with vertical hinged or sliding doors. Concrete runways (16) should be at least 1 in. above yard level and 18 in. wider than the racks or cars. Sufficient turn-

ing radius for the cars at runway intersections should be provided. For ordinary equipment sufficient turning radius is provided by placing at each corner of intersection a triangular section of paving extending 6 ft., back from the corner in both directions. This may seem like a small point in making the original layout but it is very important and unless it is provided for will lead to endless difficulty and waste of time later.

B. MATERIALS

10. Cement

Cement should meet the requirements of the current standard specifications for portland cement of the American Society for Testing Materials. It should be stored where it is completely protected against moisture.

11. Aggregates—General Requirements

Aggregates should consist of sand, pebbles, crushed stone, slag, burned shale, cinders or other approved inert materials or combinations thereof. Dense aggregates, such as sand, pebbles, crushed stone, crushed slag should be sound and durable and free from excessive amounts of dust, soft or flaky particles or shale or other deleterious materials.

Porous aggregates, either natural or manufactured should be free from injurious amounts of acid or alkali or compounds forming acids or alkalis, industrial wastes and, in the case of cinders, combustible matter in excess of 35 per cent of the weight of the aggregate. They should be of such strength as to produce units meeting the A. C. I. specifications. All aggregates should be free from frost or lumps of frozen material. Where stationary aggregate bins are provided, a coil of steam pipe should be arranged around the outlet of the bin so that the aggregates can be heated to remove frost and frozen lumps. Such heating will aid in the early hardening of concrete in cold weather. The use of crusher-run or bank-run material where the proportion of fine to coarse varies is usually unsatisfactory. The aggregates should be screened into fine and coarse particles and stored in separate bins.

12. Fine Aggregate

Fine aggregate should be considered as material passing a screen with four meshes per linear inch. It should have at least 15 per cent retained on a No. 8 screen.

13. Coarse Aggregate

Coarse aggregate should be considered as material retained on a screen having four meshes per linear inch. Maximum size of coarse aggregate should not exceed one-third the thickness of the thinnest web of the units to be manufactured.

Discussion—Storage bins should be designed to minimize segregation of aggregates. This segregation is limited to some extent by separating the materials into fine and coarse.

14. Water

Mixing water should be free from injurious amounts of oils, acids, organic material or other harmful matter. Water fit to drink is usually satisfactory.

15. Proportions

In the selection and combination of fine and coarse aggregate and in the proportioning of total aggregate to cement, the manufacturer has control to a large extent over the properties of the resulting concrete.

Discussion—The combination of fine and coarse aggregates which will develop the maximum strength may lead to units that have too rough a texture and edges that are too uneven to make the product saleable. The grading that might result in the maximum fire resistance might not be best for maximum strength or minimum absorption. For this reason, the combination of aggregates and the proportion of aggregate to cement are usually a compromise which will produce to the highest degree all of the desirable characteristics in the finished product.

16. Strength

The aggregate and the cement should be proportioned in such a way that the finished units will meet the American Concrete Institute Specifications or local building code requirements when made under local plant conditions. The only reliable way that the Committee knows for establishing definitely the proper proportions of fine to coarse aggregate and total aggregate to cement to obtain the required strength is through strength tests made in a properly equipped testing laboratory. These tests should be run frequently before the plant gets into regular commercial production and after that only often enough for checking.

In general, the coarser the grading of the aggregate, the greater the strength of the unit for any given cement content, provided there are sufficient fines in the aggregate to make the mix work-

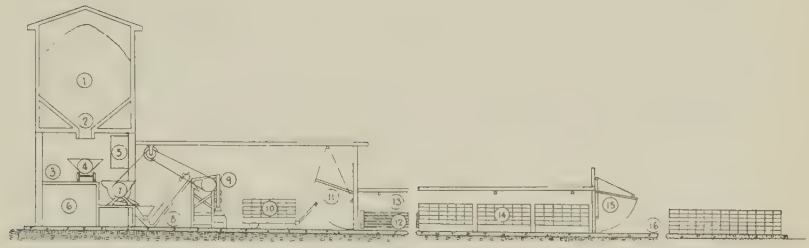


FIG. 3—TYPICAL DIAGRAMMATIC LAYOUT OF PRODUCTS PLANT

able with the given type of molding equipment. However, the grading must not be too coarse or there will be breakage in handling the units from the machine and the surfaces and edges may be too rough and irregular for a saleable product.

With the fine aggregate all passing a No. 4 sieve and the coarse aggregate graded from ¼ to ⅜ in., the maximum proportion of coarse material that can be used to advantage is about 50 per cent of the total aggregate used in the mix. Generally not more than 40 per cent of coarse material gives the desired combination and in very few instances will only 25 per cent of coarse aggregate produce a unit which has the greatest strength for a given amount of cement. The proportions of cement to be used for any given combination of fine and coarse aggregates can only be determined by trial. Trial mixes containing 25, 35, 45 and 55 per cent of coarse aggregate graded from No. 4 to ⅜ in., mixed with the fine aggregate should be made with different proportions of cement to total aggregate 1:6, 1:7, 1:8, 1:9, 1:10 and 1:11 by damp loose volume of mixed aggregate.

The first indication of the suitability of aggregate combination and cement-aggregate proportions is determined at the block machine. If the appearance of the surface and edges is satisfactory, and if the block can be handled easily and quickly without broken webs or corners, the mix is suitable with regard to appearance and handling.

After setting aside the first two and last two block from each of these trial mixes, three block from each mix should be selected

and cured according to ordinary plant practice and kept for testing at 28 days by recognized testing laboratory. From this original group of tests the manufacturer can tell within a relatively narrow margin the proportions of aggregate to cement that are necessary to produce the strength required by the A. C. I. specifications or the local code.

Uniformity of grading of aggregates is important. If the aggregate is obtained from well established commercial plants usually little difficulty is encountered. However, if the grading is irregular, it may be necessary for the manufacturer to find a different source of supply or install a set of screens to divide the material into fine and coarse and to eliminate the over-size particles.

A screen analysis of the aggregates finally selected for producing the desired units should be made and checked occasionally to determine that the grading has not varied appreciably. If it does vary adjustments in the fine and coarse aggregate proportions or in the total aggregate proportions to cement will have to be made.

It is well for a manufacturer to aim at from 75 to 100 lb. per sq. in. of gross area more strength than minimum requirements.

Discussion—In washed fine aggregate it frequently happens that too much of the very fine sand is washed away. This is particularly true of the sizes passing a 28-mesh sieve. A close distinction should be made between silt and a fine sand.

In order to obtain proper workability with dense aggregates the fine aggregate should have not less than 3 per cent nor more than 10 per cent of the material passing a 100-mesh sieve, provided the fine aggregate is well graded between the 100-mesh and ¼-in. screens. If the units in the trial batches break easily when being handled from the machine to the rack and from the rack into the curing room, the probability is that the fine aggregate is deficient in fines. A screen analysis should be run. If deficient in fines, the addition of small amounts of fine sand, stone screenings or other finely divided aggregate will aid in making the concrete workable and easily handled.

By following the suggestion above to make trial batches and have tests made the plant can get into operation on a satisfactory basis in a short time. If it is necessary to get in production in less than 28 days, block can be made and cured in the regular way and tested at 7 days. The 7 day strength can be assumed to be about 70 per cent of the 28-day strength. This is only an approximation and should be checked at 28 days.

Fine material generally cuts down the strength of the concrete and results in higher costs. Very fine material should be limited to the bare requirements

to make a block that can be handled away from the machine without cracking or breakage.

Manufacturers who use the Inspection and Certificate Service of the Underwriters' Laboratories[2] have a positive guarantee that their product will meet the A. C. I. specifications. This Service usually eliminates the necessity of further commercial tests.

17. Absorption

In combining the aggregates and in proportioning the total aggregates to cement to obtain units which meet the absorption requirements of the specification, but little difficulty is encountered if the aggregate does not contain too much or too little fines. Generally the mix that gives the maximum strength will tend to give the minimum absorption. If the absorption is too high a study of the aggregates giving particular attention to the fines will generally show what is necessary to make the correction.

18. Surface Texture

The requirements for surface texture will be controlled largely by local market conditions. Rough textures are usually desired for partition units and those that are to receive stucco or plaster. A close texture is desired for exposed ashlar and other exposed units.

Discussion—With regard to surface texture there is a wide variation in what is considered saleable in different markets. For example, in one market a very coarse textured block is acceptable to the architects and contractors even for exposed work, whereas in another market less than 100 miles away a coarse texture will not be permitted in the backing of a faced block. Keeping this in mind, the manufacturer should educate the builders from the beginning to use relatively coarse textured units.

19. Density

With the use of dense aggregates the proportions which will produce the densest products are generally those which contain the maximum amount of coarse aggregate and still contain enough fine aggregate to produce a smooth surface. With porous aggregates used in the production of light weight units, the amount of material in the mix passing a 50-mesh sieve is generally limited and in addition more of the coarse aggregate is used to produce a unit of less density and lower weight. This is generally

[2]Underwriters' Laboratories, 207 East Ohio Street, Chicago, Illinois.

desirable for light weight units except where fire resistance or water tightness are important.

20. Consistency

Concrete units made from dense aggregates on tamp or pressure machines should be mixed as wet as practicable and still prevent appreciable sagging or distortion of units. Generally a slight water web should show on the surface of the unit next to the mold box. With the pressure method a slight excess of moisture should be forced from the unit when the full pressure is applied. When vibration is used to consolidate the material the mix should be as dry as practicable to completely fill the molds and produce a pronounced water web on the surface without slumping or sagging. Where the wet cast method is used the water content should be reduced to a minimum and vibration and spading used to facilitate getting the concrete into the molds without forming aggregate pockets.

With porous aggregates, mixes somewhat drier than those used for dense aggregates are generally recommended for tamp or pressure method as this will produce a fairly coarse texture which is desired for the direct application of plaster or stucco. If porous aggregates are used in the vibration of cast methods, the surface appearance will be nearly the same as when dense aggregates are used.

21. Mixing

In dense aggregate concrete the cement and aggregate should be mixed for one minute or until the mass is uniform in color before the addition of the mixing water. All materials should be mixed from three to six minutes after the addition of the water. The longer mixing time increases the strength, density and workability. The mixer should be of such capacity that the concrete can be mixed for the time recommended without retarding production. Excessively long mixing periods, for example, from 10 to 15 minutes, while somewhat beneficial, are not generally recommended or considered economical. With porous aggregates, due to their relatively high absorption, one-half to two-thirds of the total mixing water required should be mixed with the aggregates for 2 to 3 minutes prior to the addition of the cement. Then the cement and the remaining mixing water should be added and the mixing continued for 2 to 3 minutes.

Discussion—For several years it has been considered that a batch mixer of sufficient size to keep the block making machine running at capacity with one batch every 10 minutes was necessary. The Committee recognizes that thoroughness of the mix and not time of mix is the important consideration. If, through the use of continuous mixers or other mixing methods, concrete of equal or superior strength can be obtained in less time than it has indicated, such methods should be thoroughly satisfactory.

22. Curing

Concrete hardens in the presence of moisture and heat. If moisture is not present the chemical reactions in the cement stop. If the temperature falls below 40 deg. F. these reactions practically stop. Consequently, in the absence of moisture or heat above 40 deg. F. there is little if any hardening of the concrete.

From a manufacturing standpoint it is important that units be made available for market as soon as possible. This requires that special attention be given to providing moisture and heat. The curing room should be provided with equipment that will keep the moisture in the room slightly above the saturation point at any temperatures used. Methods for obtaining this moisture were discussed in section 8.

The higher the temperature of the curing room above 40 deg. F. the more rapidly the concrete hardens, provided there is always a slight excess of moisture in the room. Means of providing artificial heat for curing were discussed in section 8. The units should be put into a curing room and protected against draft immediately after they are molded. If they are to be steam cured, the heat and moisture should be turned on as soon as possible. Proper curing conditions during the first 48 hours after the block are molded are particularly important in producing strength and density.

Discussion—In curing rooms except where high pressure steam is used it is difficult to get the temperatures much above 140 deg. F. because of heat losses. Temperatures in high pressure steam curing vary from 325 to 400 deg. F.

It has been common practice to provide a substantial excess of moisture in curing rooms. This can be eliminated because only a slight excess of moisture is required. The right amount can be determined easily by the appearance of a light fog in the curing runnel when the temperature is at its high point. Adjusting the petcocks to permit the right amount of moisture should be done when the plant is first started and little if any further adjustment will be required.

23. Length of Time of Damp Curing

With artificial heat and moisture provided as discussed in Types A., B, C and D in section 8, 24 hours of curing is considered the minimum. With high pressure steam curing, such information as is available indicates that 12 hours of curing is ample. Where only sprinkling is used the units should be protected from temperatures lower than 50 deg. F. and kept wet for not less than 3 days.

If the units are cured as described, they may be stacked in the yard and kept damp until they develop the strength and absorption requirements of the specification.

24. Dry Curing

Then the sprinkling should stop and the units should be allowed to dry. It is desirable that a covered yard or some other method be provided to keep rain and other precipitation away from the block.

Discussion—It must be recognized that the drying after wet curing is just as much a part of satisfactory manufacturing as mixing, tamping or wet curing. Several recent specifications require that when units are delivered on the job they shall not contain more than 30 to 40 per cent of the amount of moisture they hold when totally saturated. Stacking units close together in large piles prevents them from proper drying and even in the middle of the summer, units a few courses in from the outside of the pile remain damp for several months. For this reason the piles should be made narrower and it has been suggested that the block be stacked in the pile with cells horizontal and opposite so that there will be direct air currents through the pile. Some manufacturers have suggested that raising the lower course of block a few inches off the ground on small sills and then stacking the block with cells vertical and opposite will cause a chimney action to occur which will aid in drying the block. Which of these two methods is better has not been determined but either of them will aid in drying the units.

25. Faced Units

Concrete masonry units with special facing material made from selected aggregates, colored cement or other materials should be made in the following manner: The facing material should be screened to pass a $\frac{1}{8}$ in. screen and all retained on a 100-mesh screen. A mixture of 1 part of cement to $2\frac{1}{2}$ or 3 parts of aggregate will give best results. Mixes richer than these are likely to produce crazing. Mixes leaner are likely to be too absorbent. If special aggregates are used in the facing material,

as soon as the block are molded the surface is treated with a fine water fog spray to remove the surface film of cement and the units are put in the curing room and handled the same as other units.

Appendix

Report of Committee 708, February 1931

Standard Specifications for Concrete Building Block and Concrete Building Tile—(P-1-A-29)

GENERAL

1. The purpose of these specifications is to define the requirements for concrete building block and concrete building tile to be used in construction.

2. The word "concrete" shall be understood to mean portland cement concrete.

3. The average compressive strength of 3 or more concrete block or concrete building tile in lb. per sq. in. of gross cross-sectional area as laid in the wall shall not be less than 700 lb., no one unit falling below 600 lb. 28 days after being manufactured or when shipped.

Wherever concrete block or building tile are used to carry unusual heavy loads, the average compressive strength of 3 or more of these units 28 days after being manufactured or when shipped in lb. per sq. in. of gross cross-sectional area as laid in the wall shall be at least 10 times the figured superimposed load to be applied.

Non-load-bearing concrete block and concrete tile shall have sufficient strength necessary to prevent excessive breakage during delivery and handling.

4. The gross cross-sectional area of a one-piece concrete block or tile shall be considered as the product of the length times the width of the unit as laid in the wall. No allowance shall be made for air spaces in hollow units. The gross cross-sectional area of each unit of a two-piece block or tile shall be considered the product of the length of the unit times one-half the thickness of the wall for which the two-piece block or tile is intended.

5. The compressive strength of the concrete in units of all classifications except "non-load bearing block" shall be at least 1,000 lb. per sq. in., when calculated on the minimum cross-sectional area in bearing.

Absorption Requirements

6. Concrete building block and tile to be exposed to soil or weather in the finished work (without stucco, plaster or other suitable protective covering) shall meet the requirements of the absorption test.

7. All concrete building block and tile not covered by Paragraph 6 need not meet an absorption requirement.

8. Concrete block and tile shall not absorb more than 10 per cent of the dry weight of the unit when tested as hereinafter specified, except when it is made of concrete weighing less than 140 lb. per cu. ft. For block or tile made with concrete weighing less than 140 lb. per cu. ft. the absorption in per cent by weight shall not be more than 10 multiplied by 140 and divided by the unit weight in pounds per cubic foot of the concrete under consideration.

Sampling

9. Specimens for tests shall be representative of the commercial product of the plant.

10. Not less than three and preferably five specimens shall be required for each test.

11. The specimens used in the absorption test may be used for the strength test.

METHODS OF TESTING

Absorption Test

12. The specimens shall be immersed in clean water at approximately 70 deg. F. for a period of 24 hours. They shall then be removed, the surface water wiped off, and the specimens weighed. Specimens shall be dried to a constant weight at a temperature of from 212 deg. to 250 deg. F. and re-weighed. Absorption is the difference in weight divided by the weight of the dry specimens and multiplied by 100.

Weight of Concrete

13. The weight per cubic foot of the concrete in a block or tile is the weight of the unit in pounds, divided by its volume in cubic feet. To obtain the volume of the unit fill a vessel with enough water to immerse the specimen. The greatest accuracy will be obtained with the smallest vessel in which the specimen can be immersed with its length vertical. Mark the level of the water, then immerse the saturated specimen and weigh the vessel. Draw the water down to its original level and weigh the vessel again. The difference between the two weights divided by 62.5 equals the volume of the specimen in cubic feet.

Strength Test.

14. Specimens for the strength test shall be dried to constant weight at a temperature of from 212 deg. to 250 deg. F.

15. The specimens to be tested shall be carefully measured for overall dimensions of length, width and height.

16. Bearing surfaces shall be made plane by capping with plaster of paris or a mixture of ½ portland cement and ½ plaster of paris, which shall be allowed thoroughly to harden (from 3 to 6 hours) before the test. No point on the surface shall deviate from the plane more than 0.003 in. The cap shall not be thicker than ⅛-in. It shall be formed by means of an accurately machined metal plate or a heavy plate glass having a true surface.

17. Specimens shall be accurately centered in the testing machine.

18. The load shall be applied through a spherical bearing block placed on top of the specimen. The rate of loading after 50 per cent of the ultimate load has been applied shall not be greater than that which will produce a shortening of the specimen of 0.02 in. per minute.

19. When testing other than rectangular block or tile care must be taken to see that the load is applied through the center of gravity of the specimen.

20. Machined-steel or cast-iron plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and

the specimen. In no case shall the distance between the edge of the spherical bearing block and the end of the bearing plate be greater than twice the thickness of the plate. Where a number of thin plates are used, in no case shall the plates be less than one inch thick nor shall any plate extend beyond the one immediately above it a greater distance than twice the thickness of the plate.

21. The specimen shall be loaded to failure.

22. The compressive strength in pounds per square inch of gross cross-sectional area is the total applied load in pounds divided by the gross cross-sectional area in square inches.

CONVENTION DISCUSSION

W. D. M. Allan—I should like to call attention to the importance of the last point Mr. Woodworth made regarding the drying of the units after they have been cured. A unit 28 days old or 90 days old may not be any drier than one three or four days old. Several products manufacturers and others have had an opportunity in the last few weeks to criticize the master specifications of the federal government covering concrete masonry units. That specification requires that a unit shall not have more than 40 per cent of the total saturated weight of moisture when delivered on the job. That means, for instance, that a cinder concrete block with an absorption of 16 per cent when saturated, would be cut down to two-fifths of that—about 6.4 per cent, when delivered on the job. Even in the middle of the year, taking block out of the center of a large stock pile that has been rained on a month before, block will rarely meet that requirement, so we have got to add to our manufacturing processes another step that is as important as anything we have been considering in the past, and that is drying the units after they have been wet-cured. It is as certain as can be that after all the discussion on volume change and this specification covering moisture content when units are delivered, it will be a matter of only six months or a year when many specifications will have a limit of that kind. The tendency is in that direction, and every manufacturer is going to have to recognize it if he is going to get important work.

Benjamin Wilk—There are just a very few points, I think, to be considered. In view of the fact that this curing condition is so essential, it will be desirable to consider the use of paved yards entirely. We found over a considerable area of our place that a three inch paving was satisfactory. We have had such

pavement in use now for five years and it is saving us a whole lot of money. The maintenance of a gravel or cinderized yard is considerable, especially with trucks running across it, and we put in three inches of concrete which has stood up very nicely. It saves in the cost of piling. Instead of having driveways alone and instead of carrying block say, six feet, you can take your truck and bring your blocks up to the exact point where you want to pile it, and the man can take the block off the truck and pile it on to the regular pile. It will also in many cases, make a better pile, for it will often be difficult to pile block nine or ten high on cinders, gravel etc., if you want to separate them in piling. Under ordinary conditions one tier of blocks will support the next tier, but if you separate them it will be necessary to have a very even footing, and a paved yard would eliminate the trouble from block falling over.

It seems to me that consideration should be given to the fact that in using lift trucks wider curing rooms are necessary, especially in view of the fact that the pipes are on the inside. I notice it was twelve inches from the edge of the rack. A block may overhang about four inches and you only have left about eight inches. Four inches of that space may be used up with your piping and you have but four inches left for clearance. It is very difficult for the man operating the lift trucks to set the racks in such a way that he can have easy access. As far as piling block is concerned, you must take into consideration the difference between using block or tile for back up, and block or tile used for exteriors. A block piled horizontally will generally be scraped as the men place the block horizontally and as they are taking it from the pile, they naturally pull them. You may tell them a thousand times to lift them, but they won't do it, and as they pull them, little particles of sand will start scratches on the block.

Readers are referred to the JOURNAL *for October, 1931 (Vol. 28) for further discussion which may develop. Such discussion should reach the Secretary by Sept. 1, 1931.*

Discussion of Paper by C. A. Hughes and A. S. Levens:

"Strength and Shrinkage of Mortars Made with Blends of Portland Cement and Pozzuolanic Materials"*

BY LEWIS H. TUTHILL†

Several years association with the problems of mass concrete have strengthened the conclusion that plain portland cement concrete has serious deficiencies when used in this manner. More than two years ago this led to a beginning of studies of the effects of pozzuolanic co-mixtures with portland cements. About the same time contemporary work was started in three other field and industrial laboratories in California.

In many ways the paper was an excellent check on our work with Fresno pumicite, (mentioned as F in the paper under discussion) but the conclusions enumerated, although correct in the light of several independent series of similar tests, do not bring out the importance of the possibilities of the use of this and similar pozzuolanic material with portland cement—particularly in mass concrete. It is also felt that certain of the tests were over too short a period to demonstrate the ultimate action of a slow working material of this kind. This applies particularly to the strength tests, and to the shrinkage tests which were terminated just at the point where there was a decided indication that time and age materially reduced the tendency toward volume change in these concretes. In a concrete dam, for instance, it is not only what may happen in the concrete the first few months that is important, but also, its probable behavior through the first few hundred years.

*A. C. I. Journal, December 1930; *Proceedings*, Vol. 27. p. 317.

†Concrete Technologist, Los Angeles, Calif. (Exchequer, Melones, Pardee and Calaveras dams).

The use of the visual test tube method was an excellent independent check upon our method of proving the material under consideration to be a true pozzuolana. Our method was to make a water paste of two parts of pumicite to one part of hydrated lime and cast 2 x 4 in. cylinders. These specimens attained a strength of 400 lb. per sq. in. in a month while pastes of the individual ingredients mixed separately remained soft. Both of these methods and results satisfy the requirements of the accepted definition: "A substance is said to be pozzuolanic when, while not necessarily cementatious by itself, it possesses constituents which will combine with hydrated lime at ordinary temperatures in the presence of moisture to form stable insoluble compounds of cementatious value."

Compression tests of concrete made by the writer are submitted to show the effect at one year. The early lag and later catching up in strength is the same for the concretes as for the mortars discussed in the paper. Richer and drier concretes, and concretes of lower fineness modulus, show the same trend of strength increase but with an increasing period (from six months to two years) before the equivalent of the concrete in which no pumicite is substituted for cement is reached. This lag in strength is not a matter for concern where high early strength is no object as long as its ultimate attainment is assured—and it is assured, for pozzuolanic concretes show far more consistent and uniform time increase strength ratios than do the plain cement concretes. As long as the practical minimum strength necessary safely to sustain early loads is attained, the delay in reaching equivalent or usually superior ultimate strength is unimportant compared with the advantages of pozzuolanic concretes in reduction of heat liberated on setting, in fixing in an insoluble form the usually soluble lime compounds, and, particularly, in consequent durability and permanence of the structure.

Aside from establishing the fact that concretes, in which the pumicite is substituted for a portion of the cement, will attain the same or greater strength than the plain concretes with greater uniformity and without retrogression, it is submitted that in the above tests is an indication of the pozzuolanic char-

EFFECT ON COMPRESSIVE STRENGTH AT VARIOUS INTERVALS OF SUBSTITUTING FRESNO PUMICITE FOR CEMENT BY VOLUME

Per Cent Pumicite	Actual Cement bbl./c.y.	Actual Water-Cement R.*	Slump	Fineness Modulus	Strength 28 Days†	Strength 3 Mos.†	Strength 6 Mos.†
None	.94	1.09	6–8	6.57	1455	1600	1775
					1.00	1.10	1.22
10	.85	1.13	6–8	6.57	1302	1679	1867
					1.00	1.29	1.43
20	.76	1.16	6–8	6.57	1098	1469	1713
					1.00	1.34	1.56
30	.67	1.19	6–8	6.57	996	1477	1772
					1.00	1.48	1.78

*In addition to water here indicated, water was added for absorption of the aggregates and an amount equivalent to 60 per cent of the weight of the pumicite.

†Each strength value is the average of two 8 x 16 in. specimens, each from a different batch assembly. One specimen for each period was made from each batch. Aggregate 3 in. maximum.

NOTE: It was plainly noticeable, particularly in this wet consistency, that as the amount of pumicite substituted increased, the mixture became harsher as to workability. This is believed due to the lower absolute volume and mass in an equivalent volume compared with cement which reduces the lubrication. In order that workability be maintained, particularly where the remaining cement is less than 1 bbl. per cu. yd., substitution by weight is recommended.

acter: combination of lime and silica to form the strength giving (as well as insoluble) calcium silicates. For instance, in the first series the ratio of increase, 28 days to 6 months, for plain concrete is 1.22; for 30 per cent substituted pumicite it is 1.78—an increase of .56. Where does this extra margin of strength come from? Certainly not from the aggregate. The cement has gone its limit at 1.22. Free lime and soluble silica are present. Cement chemists generally agree that calcium silicates are the strength giving elements of cement. It would seem evident that some such combination of these two ingredients had been formed.

As a matter of fact all contemporary work on this subject has uniformly and consistently proven the characteristic pozzuolanic reaction with free lime of this pumicite and other available American materials. The existence of a percentage of free lime in all cements—varying in amount from brand to brand—and

EFFECT ON COMPRESSIVE STRENGTH AT VARIOUS INTERVALS OF SUBSTITUTING FRESNO PUMICITE FOR CEMENT BY WEIGHT

Test No.	Per Cent* Fresno Pumicite	Actual Cement bbl./c.y.	Maximum Size Aggregate	Slump	Fineness Modulus	Strengths Are Average of Two Tests			
						28 Days	3 Mos.	6 Mos.	1 Year
WP5	None	1.16	1½ in.	5	5.35	2082	2610	2888	2907
						1.00	1.253	1.387	1.396
WP4	14	1.00	1½ in.	5	5.35	1866	2438	2746	2978
						1.00	1.306	1.472	1.596
GT2	14	1.00	1½ in.	5	5.35	1970	2578	2870	2984
						1.00	1.308	1.456	1.514
AD2	None	1.111	1½ in.	6–7	5.61	2120	2548	2470	2442
						1.00	1.202	1.165	1.152
AD6	10	1.00	1½ in.	6	5.61	1906	2474	2579	2494
						1.00	1.297	1.35	1.307
AD4	10	1.00	1½ in.	6	5.61	2030	2434	2090	3146
						1.00	1.198	1.32	1.548
AD3	None	1.25	1½ in.	6	5.61	2998	3271	3477	3109
						1.00	1.09	1.16	1.036
AD5	20	1.00	1½ in.	6	5.61	2388	2735	2862	3242
						1.00	1.144	1.19	1.357
AD7	20	1.00	1½ in.	6	5.61	2048	2643	2955	3275
						1.00	1.289	1.44	1.598
AD1	None	1.00	1½ in.	6	5.61	2032	2348	2360	2323
						1.00	1.156	1.162	1.144
21	12.5	1.19	1½ in.	5–7	5.35	2537	3060	3230	3515
Job						1.00	1.206	1.263	1.385
Tests									

*The percentage of pumicite is the percentage of the cement plus pumicite.

NOTE: In all the series AD two year specimens were made. Tests will be made on these at the proper time in the fall of 1931.

the disastrous effect of its solubility in the dissolution and decay of hydraulic and sea water structures in apparently recognized in this country as evidenced by numerous articles and references on this phenomenon. But so far, indirect rather than fundamental measures have been generally taken to combat these results. We must confess that the Europeans are somewhat in the lead of us in the solution of this problem by the recognition of the effect of using pozzuolanic co-mixtures with their portland cements. Some quotations from European publications and statements in limited circulation are believed to be pertinent and instructive.

From Paper No. 693—"Scientific Researches on Pozzuolana Substances," by A. M. Manonilov, Director of the Mechanical Laboratory of the Moscow Institute of Transport Engineers:

The scope of the question may easily be gaged from the fact that 14 papers, bearing on the decay of portland cement in water, were delivered in English, Swedish, Danish, Dutch, Belgian, French, Spanish and Japanese experts during the last International Congress on Navigation held in London in 1923.

All these numerous scientific labors in investigating the decay of portland cement in water have definitely shown that the cause of this decay is not to be assigned to faulty work in building concrete structures; nor to faulty portland cement used in the cases in question; but is inherent to the very nature of portland cement, whose process of setting involves the formation of a quantity of hydrate of CaO (which is $Ca(OH)_2$ easily subject to solution in water and to gradual alkalization. These researches have conclusively shown that it is to this particular process of alkalization of the $Ca(OH)_2$ that the weakening of concrete and its subsequent decay is due. In the presence of sulphates, usually found in sea water, the process of decay of portland cement is considerably accelerated owing to the additional chemical reactions tending to form lime sulpho-aluminate: 3Ca) Al_2O3 3Ca SO_4 $3OH_2O$. This formation of lime sulpho-aluminate results in huge increase in volume which in turn is followed by the formation of numerous cracks increasing the permeability of concrete to water and contributing to a quicker alkalization of $Ca(OH)_2$. In the presence of magnesial salts in water soluble calcium salts are formed which can be easily washed away by water.

Thus, since the law of decay of portland cement in water was established, the question naturally arose of finding means to prevent this process of destruction.

* * *

After extensive scientific investigations had been started, all the countries concerned soon came to the conclusion that the best, the simplest and cheapest method of preventing the decay of portland cement in water was that of introducing pozzuolana aggregates into the concrete, the "active silica SiO_2

being the main component of the above aggregate. The prevention of decay of portland cement in this case consists of the active silica SiO_2 binding the hydrate of calcium oxide $Ca(OH)_2$ and thus creating the combination $CaO\ SiO_2\ 2\frac{1}{2}H_2O$ which is next to insoluble in water. Another favorable factor in that the possible formation of destructive lime sulpho-aluminate $3CaO\ Al_2O_3\ 3Ca\ SO_4\ 30H_2O$ is altogether eliminated provided only a sufficient quantity of active silica SiO_2 is available.

* * *

Port structures at Odessa, Yalta, Feodossia, Novorossijsk, Poti, Batum and Petrovsk were inspected, and so also were the dry docks in Sebastopol and Baku. The piers in these ports were built after the Marseilles type. Standard portland cement was made use of in all these structures, excepting only those of Odessa and Poti where Roman pozzuolana and Teil lime were employed along with portland cement. The results obtained from the inspection of concrete structures in South Russian ports of the Black and Caspian seas led to the same conclusions with regard to the decay of portland cement in sea water which at that time had already been arrived at abroad. The inspection showed that visible symptoms of decay could be already observed seven years after immersion; a considerable degree of decay was observable in the course of 13-14 years, and complete destruction took place in 25-30 years. (where only portland cement was used).

Two pertinent conclusions were:

(1) Extensive scientific analytical work was carried out in tracing the true cause of this decay and resulted in the same conclusions which were also arrived at by scientists of other countries. The research resulted in raising full scale the question of the tremendous value of pozzuolana substances.

(2) In connection with the scientific and industrial conclusions obtained as to the expediency of applying pozzuolana substances, the question has been given these last ten years such great attention in the USSR that pozzuolana substances are incorporated in the official Matrice Standards as a substantial item thereof.

[Mr. Tuthill also refers to "Building Research Bulletin No. 2—Pozzolanas" of the Department of Scientific and Industrial Research, England, and to discussion by Prof. Lingi Luiggi, (Italy) of a paper by J. B. Lippincott describing the successful use of pozzuolanic material ground with cement in construction of the Los Angeles aqueduct, *Transactions* of the *American Society of Civil Engineers*, 1913, Vol. LXXVI, pp. 567-9.—Editor]

So it would appear from international research, experience and opinion (including our own), which conforms so well from one source to another, that it is true that:

1. Portland cement concrete is improved by the inclusion of an intimately mixed pozzuolana by reason of its combination with otherwise free and soluble lime to form insoluble and enduring compounds.

2. From centuries of experience of the durability of lime-pozzuolana compound mortars and concretes, it may be taken for certain today that there will be certainly no decrease, and most likely an increase in the durability and permanence of concretes in which some pozzuolana of proven reaction with lime is used as a co-mixture with portland cement.

3. Available American materials are equal or superior to similar materials used so widely abroad with profit and success, and otherwise unusually permanent results.

All that has been accomplished to date in volume change studies is a classification of causes and effects, and a recognition of the factors increasing and those decreasing this unpleasant phenomenon in concrete—particularly in massive work.

It is evident that with present day materials and the best of practice, volume change cannot be eliminated. Fortunately the best practice to other ends results in minimum volume change (avoidance of excess water and fines, and thorough curing). It should be admitted that volume change is a property of concrete which must be recognized and provided for in design the same as its strength or its weight.

In this light, any slight variation more or less in volume changes caused by the use of any particular ingredient is not important. It is a property that should be otherwise recognized and properly taken care of in design, and if this is properly done, variations of a few per cent would be of no actual importance—especially when attended by such important improvements in the other characteristics of the concrete as described above. Although Fresno pumicite (like any other extremely fine ingredient) slightly increases shrinkage on drying, structures in which this pozzuolana was used show less cracking than other structures in similar circumstances but made with plain concrete. This may be because pozzuolanic concrete is enough more elastic to offset any increment of increased shrinkage factor. Experiments have shown a ten per cent reduction of modulus of elasticity for such concretes.

BY F. M. LEA*

Messrs. Hughes and Levens refer to the method of the Building Research Board for estimation of the activity of pozzuolanas.

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This method has now been replaced by a more accurate quantitative method.[1] The pozzuolana is ground to pass a 180 mesh to the inch sieve and its rate of reaction at 25 deg. C. with a N/30 $Ca(OH)_2$ solution, when mixed in the proportion of 0.5 gm. solid to 125 cc. solution, measured by an electrical conductivity method. Initially an adsorption of $Ca(OH)_2$ takes place, but this adsorption bears no relation to the pozzuolanic activity of the material. It is followed by a steady combination reaction between lime and pozzuolana and the mean rate of this action between the twentieth and forty-fifth hours after mixing is defined as the activity of the material. The activity figure so obtained is a good index of the pozzuolanic properties of the material at early ages up to about 28 days, but it does not afford any measure of, for instance, the strength which will be developed at longer ages.

Some typical activity figures are shown in the following table.

RELATIVE ACTIVITIES OF POZZUOLANAS AT 25 DEG. C.

Pozzuolana	*Activity.* (mgm. $Ca(OH)_2$ removed per 1000 mins. by 0.5 gm. pozzuolana from 125 cc. $Ca(OH)_2$ solution).
Burnt clay A	7.2
B	10.5
C	6.0
D	3.3
Scottish spent oil shale A	7.5
B	6.0
C	5.0
Trass A	2.9
B	1.2
C	4.7
Santorin earth A	2.9
B	1.1
Italian pozzuolana	2.4
Granulated blast furnace slags	0–3

The two naturally occurring volcanic rocks F and P studied by Hughes and Levens appear from their results not to differ widely from the European naturally occurring pozzuolanas such as trass and Santorin earth. Extensive studies on the production and properties of pozzuolanas which have been in progress for some years at this Station have shown that artificial pozzuolanas

[1] c. f. Annual Reports of the Building Research Board, 1926-1929.

can be produced which are markedly superior to the naturally occurring materials. Such materials are selected clays burnt at predetermined and controlled temperatures, and the spent oil shale of the Scottish shale oil industry. These materials are ground to a residue of a few per cent on a 180 mesh to the inch sieve for use as pozzuolanas. It is doubtful if granulated blast furnace slag is to be regarded as a pozzuolana in the strict sense of the term since this material does not combine very appreciably with free lime in set mortars or concretes.

The properties of a pozzuolana may be determined and compared in a number of ways, but the most useful are initially an activity test by the conductivity method mentioned above followed by tensile strength tests on lime-pozzuolana-sand briquettes stored under carbon dioxide-free conditions. These tests can then be followed, if results warrant it, by tensile strength tests on cement-pozzuolana-sand mortars and compressive strength and other tests on concretes. Measurement of the free lime present in set cement-pozzuolana mortars and concretes is another important and valuable method for assessing the value of pozzuolanas. Suitable methods for these determinations have been developed.[2] The substitution of 40 per cent cement by an active pozzuolana of the spent oil shale or burnt clay type reduces the free lime content of a water stored 1:3 cement-sand mortar from about 2.0 per cent to 0.8 per cent at 7 days, from 2.3 per cent to 1.0 per cent at 28 days and to about 0.7 per cent at 6 months.

The suggestion made by Hughes and Levens that prolonged moist air or water curing is necessary for the satisfactory utilization of pozzuolanas seems to apply more to the naturally occurring materials than to the artificial products. This is well illustrated by some data on the tensile strength of 1:1:6 pozzuolana-hydrated lime-standard sand mortars. The briquettes were similar to those specified in the British standard specification for portland cement except that mechanical methods of mixing and ramming were employed. This substitution of mechanical for hand methods has no very marked effect on strength. Results are the mean of six briquettes tested in the same conditions as stored.

[2]A general account of these methods will appear in Building Research Special Report No. 17 to be published shortly.

TENSILE STRENGTH 1:1:6 POZZUOLANA : HYDRATED LIME : STANDARD SAND MORTARS STORED AT 12-13 DEG. C.

Pozzuolana	Tensile Strength lb. per sq. inch									
	Dry stored					Water stored				
	7 da.	28 da.	3 mo.	6 mo.	1 yr.	7 da.	28 da.	3 mo.	6 mo.	1 yr.
Scottish spent oil shale	169	280	344	324	400	107	207	341	466	521
Burnt clay	192	296	373	391	393	155	263	337	387	400
Santorin earth	41	110	143	201	...	8	62	306	446	...

Considerably higher strengths are obtainable in lime-pozzuolana mortars in which the ratio of lime to pozzuolana is decreased while keeping constant the ratio of pozzuolana plus lime to sand. The effect of temperature on pozzuolanic activity is very large and in tests on lime-pozzuolana mortars at least careful control of storage temperatures is essential. This is particularly noticeable with the naturally occurring pozzuolana Santorin earth which gives such low strengths over short periods at 12-13 deg. C. The tensile strengths at 7 and 28 days of water stored 1:1:6 Santorin earth-hydrated lime-standard sand mortars stored at 25 deg. C. were 45 and 250 lb. per sq. inch, or some four to five times the strengths developed at 12-13 deg. C.

Hughes and Levens find large reductions in the compressive strength of 1:3 cement-sand mortars at early ages when the volcanic earths are substituted for cement. This is similar to the results found with trass. The substitution of 40 per cent cement by trass results in about 50 per cent reduction in strength at 7 days and about 40 per cent at 28 days in 1:2.6 concrete mixes at a constant slump of either ½ inch or 2 inches. In 1:5 mixes the corresponding reductions in strength are about 65 per cent at 7 days, 55 per cent at 28 days and 45 per cent at longer periods. The water:cement × pozzuolana ratio required for a given slump is increased by the substitution of trass for cement. Relatively few data are as yet available on the compressive strengths of concretes with shale and burnt clay pozzuolanas, but from preliminary results it appears that the strengths obtained are much higher than when trass is used. The tensile strength of 1:3 cement:sand mortars is slightly increased at 7 days by the substitution of 20 per cent of the cement by a spent shale or

burnt clay pozzuolana and decreased 5-15 per cent by the substitution of 40 per cent, but at 28 days the unsubstituted cement mortar strength is exceeded by 10-20 per cent in the first case and by 0-15 per cent in the second. For all amounts of substitution down to a 40 per cent cement 60 per cent pozzuolana mix the ultimate (1 year) tensile strengths of the pozzuolana-cement mortars equal or exceed those of the mortar made from unsubstituted cement.

A short account of some of the main lines of investigation on pozzuolanas that have been carried out will appear in the Annual Report of the Building Research Board for 1930.

Readers are referred to the JOURNAL *for October, 1931 (Vol. 28) for further discussion which may develop and for the authors' closure. Such discussion should reach the Secretary by Sept. 1, 1931.*

Discussion of a paper by U. T. Berg:

"Continuous Beams and Frames in Building Construction"*

AUTHOR'S CORRECTION

On page 363—Table 3

For $\frac{I_h \cdot L}{H' \cdot I} = 0$, the bracket for K_s should include P and N values, and K_d should apply to the D values only.

On page 371

$H' = H$ is incorrect for condition of loading as shown, Fig. 9B. As column moments are of constant value throughout $\phi_2 = \frac{H}{2\,E\,I}$ and consequently $H' = \frac{3}{2}\,H$.

*A. C. I. Journal, December 1930; *Proceedings* Vol. 27, p. 359.

Discussion of a paper by F. E. Richart and V. P. Jensen:

"Tests of Bonding Floor Finish to Slabs of Haydite and Gravel Concrete"*

DISCUSSION

BY JOHN G. AHLERS†

The data presented in this paper confirm experience gathered from practice in the field; that it is possible to apply cement finish to structural slabs within 24-hours and have them act as an integral part of the structure.

This paper should bring to engineers and architects the conclusion that specifications should be written so as to allow contractors to place floor finish within 24-hours of the pouring of the slab and have these considered as monolithic and part of the structural slab.

Experience in the past has shown that where careful construction methods are followed one inch finish, placed afterwards, can be made to bond thoroughly with the structural members and the data presented confirms what has been observed in practice.

As might be expected, this paper shows that there is equally good bonding between structural slabs of haydite and structural slabs of gravel-concrete.

BY M. A. TIMLIN

Messrs Richart and Jensen show definitely that there is no inherent defect in the bond of a rich floor finish to a base of Haydite concrete. It would seem at first that the bond of the Haydite concrete was better than with the gravel concrete since failure occurred in the concrete itself immediately below the bonding plane in the former and entirely in the bond in the latter. A definite conclusion to this effect can not, however, be drawn from the data available.

*A. C. I. Journal, December 1930; *Proceedings* Vol. 27, p. 339.
†Barney-Ahlers Construction Corp., N. Y. C.

It would be difficult if not impossible to evaluate the strength of the bond from these tests. In the gravel concrete and the floor finish the moduli of elasticity were nearly enough the same that the slabs would act practically as simple beams with relatively low shear stress between the finish and base course.

On the other hand, with a finish having a modulus of elasticity of 4,030,000 lbs. per sq. in. superimposed on a Haydite base course with a modulus of elasticity ranging from 920,000 to 1,860,000 lbs. per sq. in., depending on the proportions used, when the load is applied to the slab the Haydite concrete tends to compress throwing greater load into the finish course. When this occurs, for each unit of load applied, greater stress exists in the finish course and less stress in the base course than when gravel concrete was used and the moduli of elasticity were more nearly equal. This of course causes greater shear stress.

If failure of the bond had taken place before failure of the reinforcement it might be possible to compute what shear stress had existed at the time of failure. Since initial failure of the reinforcement occurred first this can not be done. As soon as such failure took place the concrete in the base course was immediately placed in tension while the finish was still in compression. Just what these two stresses were can not be determined and consequently the shear stress is an unknown quantity, but would undoubtedly be greater in the case of Haydite concrete and greater in the lean than in the richer mixes.

Since the finish on the Haydite slabs had pieces of the base course adhering to them, indicating failure in the Haydite concrete immediately below the bonding plane rather than in the bond itself, it would appear that the shear strength of the bond were greater than the shear strength of the concrete.

With the gravel concrete base, the shear strength of the concrete exceeded that of the bond.

Therefore if the shear strength of the concrete is a function of the compressive strength only, the bond to the Haydite concrete was better than to the gravel concrete since the richer Haydite mixture had higher compressive strength than the leaner gravel mixture. If however, the shear strength depends also on the

modulus of elasticity and other factors, as it probably does, this line of reasoning will not apply.

A point of particular interest brought out by Messrs. Richart and Jensen is that the finish course took a large part of the load, particularly in the slabs with lean base courses and should be figured as part of the slab thickness.

There are a number of questions which arise as to actual behavior in a floor which are not covered by these tests, but which should be given serious consideration. Probably the most important of these is the differential expansion and contraction due to differences in moisture and temperature conditions. This has long been recognized as an important factor where a rich finish is used on a lean base course of gravel concrete. Since this is so where the differences in the coefficients of expansion are low, it would be of far more importance between gravel topping and Haydite base where the differences in the coefficients of expansion are probably of greater magnitude. The importance of this point could be determined only by laboratory tests showing the differences in expansion of two types of concrete under varying conditions of moisture and temperature.

While we have no data bearing on the resistance to impact of a floor consisting of base course of Haydite and a rich gravel concrete finish, it is logical to assume that such resistance would be at least equal to that of a gravel concrete base. Concrete made with an aggregate having the characteristics of Haydite would serve as a cushion and be able to absorb impact transmitted to it through the finish course better than a more rigid material. There should, therefore, be less tendency to crack or to split off from the finish course.

The report under discussion is based on tests made to show the behavior of a slab having the rich finish course in compression as in the case of a slab between supports. It would be interesting to know what the action would be when the compression is taken entirely by the lean base course and the finish course is in tension, and the slab of course properly reinforced, as in the case of a slab passing over a beam or attached to a wall.

Durability of Concrete

A Report of Committee 801

A. E. LINDAU, CHAIRMAN

Two papers on the following pages—"Study of Defective Concrete," by F. R. McMillan and "More Lessons from Concrete Structures in Service," by Roderick B. Young—both members of Committee 801, Durability of Concrete—were presented at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931, with the following remarks by Mr. Lindau, chairman of the Committee.—Editor

It is my privilege to introduce the speakers on the assumption that I should know something about the committee work. You will notice that this number appears on the program as a Symposium by members of Committee 801, and that there are two who are "symposing," which probably reduces a symposium to its simplest terms. It was intended to be a more complex symposium than we have. There has been enough work done by the committee to fill a session; at one time the Program committee hoped that might be done, but other things intervened and we are privileged to have the two papers as here presented. The work of the committee has been going on for some time and it will probably go on for some time more. The committee has undertaken to survey structures individually and collectively. This survey has taken the members of the committee individually and in groups from, I was going to say, "Greenland's icy mountains to India's coral strands," but that is a little large; they did make a survey of structures though in what you might call the

sub-arctic regions of Canada, and continued at other times to the sub-tropical regions of our southern states, and from the Atlantic Ocean to the Pacific Ocean; so geographically they have done a big job. The committee is functioning by making these reports from time to time, as individual papers.

Readers are referred to the JOURNAL *for Nov. 1931, (Vol. 28) for discussion of the two papers which follow. Such discussion should reach the Secretary by Oct. 1, 1931.*

Study of Defective Concrete*

BY F. R. MCMILLAN†

In a survey of concrete structures representative of almost every kind of aggregate, method of construction, and condition of exposure to be found in the greater part of the United States and portions of Canada, a considerable amount of clinical material has been developed from which much can be learned both as to what to avoid and what to require to obtain more durable structures. While it is hoped later to make an extended presentation of the data from this survey, this paper will be limited to a discussion of some of the ways in which different causes of disintegration or unsatisfactory concrete are identified.

SOURCE OF INFORMATION

In a survey designed to bring out the causes of defective concrete, there are three principal sources which must be relied upon for information:

(1) Personal recollections of competent persons engaged on the work at the time of construction.
(2) Original records.
(3) The condition of the structure itself.

Personal recollections of responsible individuals are not often available and frequently none too reliable. Original records, when available, are most reliable as to the details covered, but good original construction records are very rare and seldom do they cover all the points upon which the concrete engineer of today seeks information. It is, therefore, the evidence presented by the structure itself upon which the investigator must to a large extent rely for his information. This would be somewhat discouraging if there were available for study only a few struc-

*Presented at the 27th Annual Convention, A. C. I., Milwaukee, Feb. 24-26, 1931, as part of the work of Committee 801, Durability of Concrete—see p. 1037.
†Director of Research, Portland Cement Association, Chicago.

tures representing a limited set of conditions, but as the scope of the study is extended the indications multiply rapidly until it is possible to read with considerable accuracy the causes of defective concrete in the present condition of the structure.

In reference to personal testimony, it is of interest to consider the fallibility of man's recollection as applied to details of events that transpired a dozen years before. Sometimes the personal evidence has the ring of reliability and accuracy and serves to illuminate and corroborate the evidence presented by the structure itself. Equally often the statements are not convincing and cannot be squared with findings on the ground. A desire to conceal or soften the facts is encountered sometimes. This, however, can usually be detected and the true condition revealed by a few judicious questions.

Not always is it the memory that is at fault when the recollections fail to agree with the evidence presented by the structure itself. Frequently, the memory is quite accurate as to things that were supposed to have been done, but the inexorable effect of time has disclosed that the work was not carried out as directed.

It may be pertinent to point out the desirability of making complete records of the construction operations. With a few outstanding exceptions like the structures of the Reclamation Service and the Crystal Springs Dam, it was the experience in this survey that the records available were generally disappointing.

EXAMINATION OF BOTH SOUND AND DEFECTIVE CONCRETE ESSENTIAL

In an examination of a structure, the portions which show no defects require study as well as the defective portions. The extent and position of the unaffected areas with relation to exposure and conditions of placement give important information. The general character of the surface and its variation from point to point may show conditions as they existed in the now defective portions at some previous stage. Similarly, the study of structures in which only slight defects are in evidence may provide essential information for the explanation of some other structure. The significance of these comments will be brought out in a

number of the illustrations of this discussion; as specific instances the cases of the XY building and of the Gull Lake and Pine River Dams are cited.

The XY building, not identified for obvious reasons, had shown serious spalling of the concrete jambs just inside of metal window frames and the frames were badly rusted. At the time of the inspection, the windows on the most exposed side of the building were so completely affected, that many had to be replaced and the concrete around the openings restored. Those that had not been reconditioned were in such advanced state that no explanation of the cause could be discovered. However, an examination of the sides of the building least exposed to wind and rain disclosed a number of the windows in which the development of the trouble could readily be traced through its various stages. This study showed that the trouble began from rusting of the sash on the inside at a point just above the horizontal pivot of the ventilating sash. These ventilating sash were in the upper portion of the window and when opened (top inward) permitted the rain, driven by the wind, to rebound and strike the inside of the frame. At some of the windows only the first signs of rust were showing, on others the rusting had progressed downward and upward varying distances with resulting spalling also in various degrees. Here was a complete record showing all the stages through which the other windows had passed—a record which could not have been added to by the original construction records, or by personal recollections of those engaged on the work.

Gull Lake Dam is a case where a study of a well preserved structure provides an explanation of defective conditions found elsewhere. This is one of five dams forming navigation control reservoirs in the headwaters of the Mississippi River, built by the U. S. Engineers Corps between 1902 and 1907. This is a group of structures in which the field notes spoke eloquently of the workmanship and supervision—conclusions which were later confirmed by personal discussion with the foreman in charge of the construction of all five dams.

Figs. 1 and 2 show three views of this dam which bring out both the excellent workmanship at the time of construction, and the perfect condition of the structure today. The complete absence of spalling at the water line is particularly noticeable. At

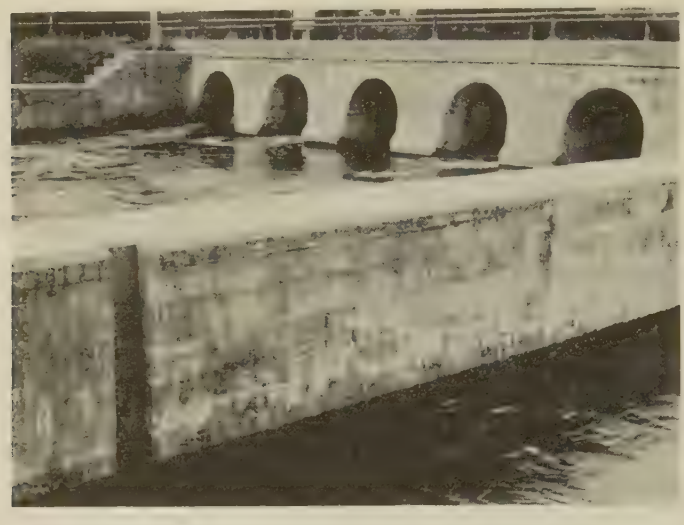


FIG. 1—GULL LAKE DAM, MINN. EXAMPLE OF GOOD CONSTRUCTION METHODS. ALMOST NO DEFECTS EVIDENT AFTER 25 YEARS

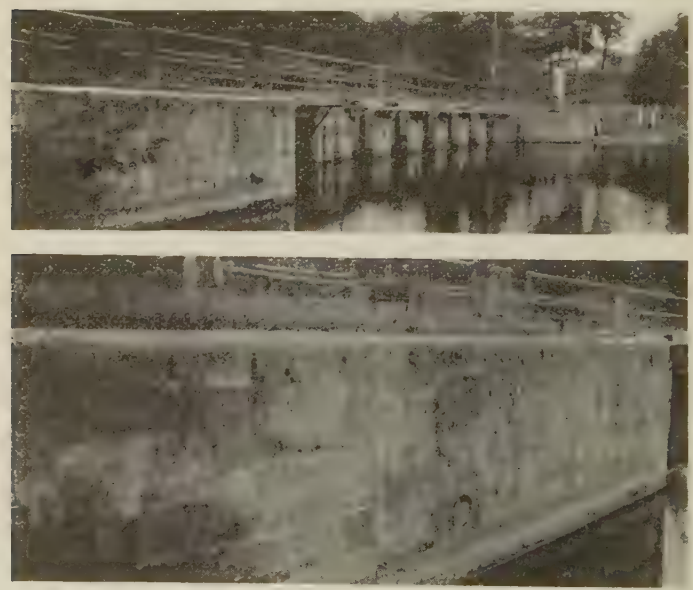


FIG. 2—DETAIL VIEWS OF GULL LAKE DAM, MINN.

the time of the inspection, it seemed important to know whether this unusual condition was due to specially good materials and workmanship, or merely to the absence of conditions which produce spalling. Such knowledge was necessary, not only to a proper understanding of this structure, but of others where such spalling is observed.

Several clearly defined water lines were in evidence (some are visible in the pictures) indicating that the reservoir level might not have been stationary long enough to produce spalling. Yet it seemed that in 22 winters in this climate some spalling would have occurred unless the concrete were unusually resistant. By close examination of the concrete below the water surface, areas were located where well-defined spalls had occurred at the corners of three of the piers. These were the characteristic V-type spalls so often found at the water surface and indicated that at some time in the past, water surface and ice conditions had been such as to produce this spalling and ravelling at the corners. The occurrence of these corner breaks at only a few places over the entire area was interpreted as showing certain defective spots in the concrete which yielded to the severe conditions, while the general excellence of the concrete was such that the disintegration was thus localized.

Another of the five dams of the Mississippi Headwaters is Pine River Dam shown in Fig. 3 in which, as in Gull Lake Dam, the presence of certain defects in an otherwise perfect structure is convincing evidence that it was the unusually good workmanship in the rest of the structure which accounts for its excellent condition after 21 years. The single defect in this dam is the calcium carbonate deposit seen in Fig. 3 on the wall of the fishway near the end where it joins the piers of the dam. This deposit shows that a slight leakage had taken place along a fill plane, forming this incrustation in exactly the same manner as similar or large deposits are formed in so many structures.

Pine River Dam has been used by the writer in a number of other discussions because it is such an outstanding example of what it is possible to obtain with careful construction methods. It was the last and noticeably the best of the group of five dams all of which were built under the supervision of Hans Olson.



FIG. 3—PINE RIVER DAM, MINN. ANOTHER STRUCTURE OF EXCELLENT RECORD DUE TO GOOD CONSTRUCTION METHODS

The method used by Mr. Olson should be made a matter of record as it is thoroughly sound in the light of our present-day knowledge of concrete mixtures, and accounts for the splendid performance which these structures have given. The proportion of sand to cement was determined by the engineer in charge by comparative tests of mortars using the available sand and standard sand. The instructions were to use with this mortar as much of the coarse aggregate as was possible and still obtain a mixture which could be worked into place, and in which there were no unfilled pockets between the coarse aggregate particles. The instructions also required placing the concrete in thin layers and thorough ramming with tampers weighing 35 lb. Only enough water was to be used to show on the surface after such thorough ramming. Mr. Olson found that the workmen were inclined to slight the tamping with these heavy tampers. By reducing these to a weight of 25 lb., he obtained much better results. He also found that he got better consolidation of the successive layers if he used somewhat more water than was intended in the instructions. When the extra water resulted in a tendency to overwetness, this was counteracted by the use of

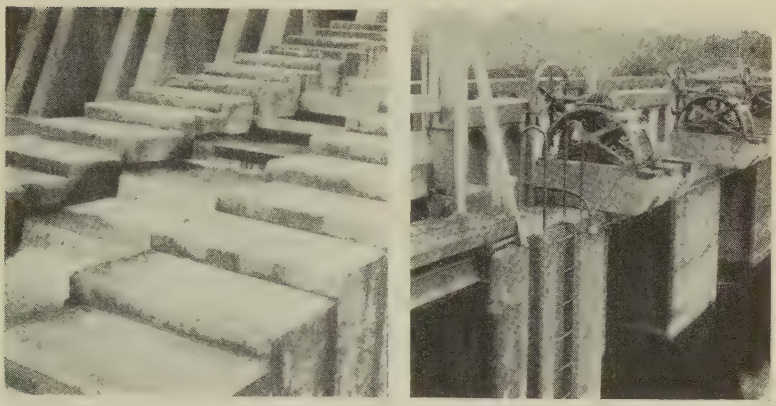


FIG. 4—DETAIL VIEWS OF PINE RIVER DAM

batches somewhat drier than the average. In this manner he obtained not only a homogeneous concrete in each layer and a thorough bonding of the layers, but avoided the undesirable accumulation of water and laitance as the placing progressed. The detail views of Pine River Dam in Fig. 4 show the excellent results from this method.

EVIDENCE OF DRY MIXES

The use of mixes too dry for proper placement or consolidation is usually quite easily detected from the appearance of the structure. The defects resulting from this cause are well illustrated in Fig. 5, which shows 2 views of the Thompson Dam near Duluth, built in 1906. This dam was described by H. C. Ash in a paper before this Institute in 1929. In the picture at the left, the failure to bond successive layers is indicated by the calcium carbonate deposits brought to the surface by water seeping along the surface between layers.

The successive layers are quite clearly marked by these incrustations. These show the irregularity which would be expected from dry mixes spread in thin layers.

The picture at the right shows two nearby sections in which the seepage between successive layers was nearly eliminated. This was accomplished by the use of a slightly wetter consistency. In the section at the left in this picture the bond between the portions



FIG. 5—THOMPSON DAM, MINN. SEE PROCEEDINGS, A. C. I., V. 25, P. 29, 1929

representing successive day's lifts was not perfect, as can be seen by the heavy incrustation of calcium carbonate below the surfaces of separation. In the section at the extreme right the bond between successive layers is perfect, and that between successive day's work is nearly so.

Mr. Ash, in his paper, makes an interesting comment on the difficulties he had in training inspectors out of the habit of insisting on too dry mixes. After the first few sections, his method, which was something like that used on Pine River Dam, was mastered and the structure completed according to his desires. The three sections to be seen in Fig. 5 show the stages in this improvement.

In considering the defects of the type illustrated in the picture at the left in Fig. 5, it must be remembered that they are principally of interest as they affect the appearance of the structure. The seepage had practically ceased years ago and the integrity of the structure has not been affected. It has been the almost universal experience in this survey of structures that where concrete placed by this dry tamped process was encountered, the structure has stood up well under all conditions even though an unsightly appearance resulted from such defects as pointed out above.

The dam shown in Fig. 6, built in northern United States in 1898, is another case where the use of dry mixtures is plainly evident. The various layers have been sharply defined by the frost action on the portion of the concrete at the lower edge of each layer. The dry, stiff mixes are difficult to work into the acute angle between the horizontal surface and the downstream face of the dam. This provides an easy point of attack from the frost action which is most pronounced at the lower edge of each layer. A lesser amount of spalling is shown at the top of each layer. The surface of this dam also shows the aggregate exposed by erosion, solvent action, and frost. Except for these surface defects this dam is in excellent condition and should continue to serve its purpose almost indefinitely.

Fig. 7 shows portions of a western dam, more than 30 years old, in a locality subject to moderate frost action. In the upper view, considerable spalling can be seen at several of the fill planes.



FIG. 6—AN EXAMPLE OF DRY TAMPED CONCRETE AFTER 30 YEARS EXPOSURE

The stiff consistency of the mixture is indicated by the steep slope of the surface marking the boundaries of successive lifts. Some undercutting where the concrete joins the rock is seen at the extreme right of the view. Aside from these defects, the entire surface is seen to be very well preserved. The discoloration is the result of a growth of moss which is quite heavy in some spots. In the lower view, the mixture was somewhat wetter as shown by the flatter slope to the fill planes, although this could not be described as a wet mix as this term is used at the present time.

It was noted above that it was the almost universal experience to find well-preserved concrete when the dry-tamped mixes were

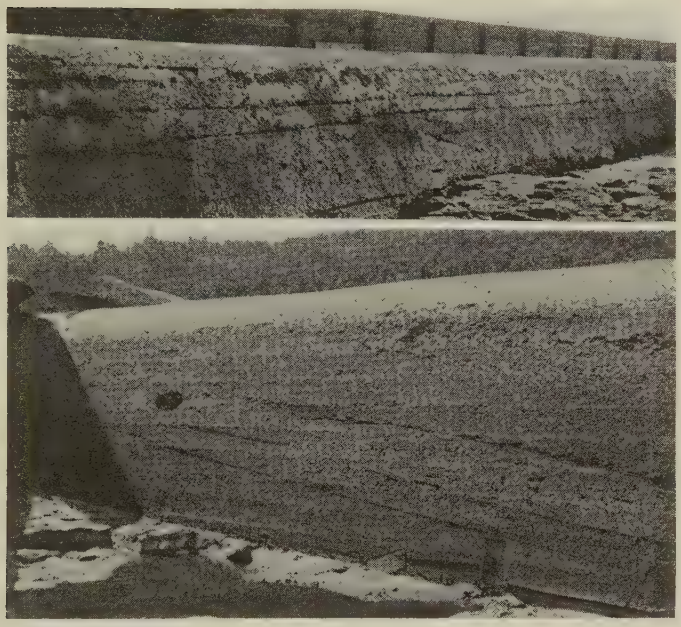


FIG. 7—ILLUSTRATING DRY AND SEMI-DRY MIXES

used. It is significant also that where mixtures of this type were used there is less confusion as to the causes of defects. It will be brought out in what follows that in certain cases it is very difficult to determine the factors primarily responsible for the condition of the structure, but this is not the common case where the dry tamped mixes were used. This rather clearly indicates that the better concrete which is likely with the dry mixes, combined with the freedom from serious segregation obtained by this method of placing, serves to protect against deterioration from other causes such as poorly graded or dirty aggregates, or even aggregate that in less durable concrete would clearly show unsound.

Fig. 8 shows another structure in which the dry tamped concrete is evident from the irregularity of the surfaces of contact and lack of bond between successive layers. This is a Lake Michigan breakwater built about 30 years ago. In the view at the bottom, the surfaces separating the successive layers are discernible through the thin mortar protection though not so



FIG. 8—A LAKE MICHIGAN BREAKWATER AFTER 30 YEARS OF SERVICE. NOTE GOOD CONDITION OF THE DRY TAMPED MIXES

readily as in the view at the top in which this thin coating is completely weathered away. Both these views show the general character of the construction resulting from this method of placing and its generally good condition after 30 years of about the severest kind of exposure.

On the horizontal surface in the lower view of Fig. 8, board marks are clearly visible. Such use of boards to protect the top surfaces of walls has been frequently encountered in structures of the period of this breakwater. Incidentally, this structure also provides a good illustration of the inaccuracy of the personal recollections of individuals in regard to details of the work. In this case the carpenter foreman who built the forms for this

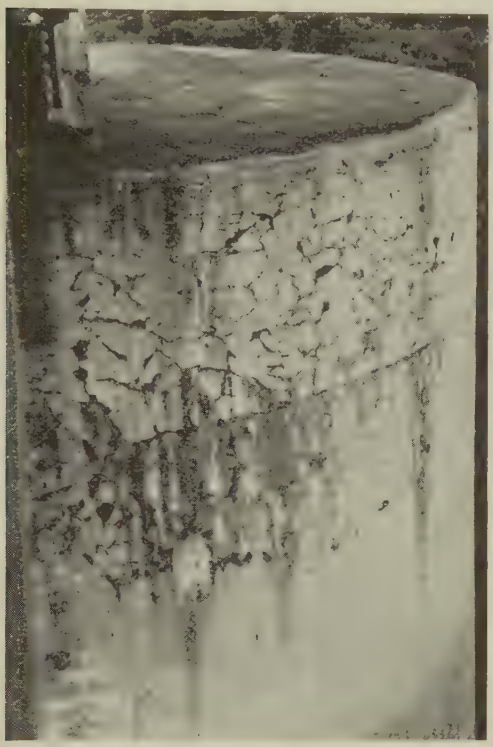


FIG. 9—AN EXAMPLE OF DISINTEGRATION DUE TO UNSOUND AGGREGATE

breakwater was interviewed and stated with some positiveness that the concrete mixtures were very wet. This recollection seems to have been based on the fact that the men wore rubber boots when distributing the concrete in the form, but it is not supported by the evidence of the structure itself.

EVIDENCE OF UNSOUND AGGREGATE

Fig. 9 shows a railway pier in which a fairly advanced state of disintegration due to unsound aggregates is illustrated. It is not obvious on first sight that the appearance of the surface in this picture is indicative of coarse aggregate disintegration. As can be seen in this picture, the lower part of the pier is still in very good condition and it is only in the upper 3 ft. that disintegration has begun. In this respect conditions are similar to those in which



FIG. 10—EXAMPLE OF AN EXCELLENT STRUCTURE MARRED BY SEEPAGE AT A FEW JOINTS

overwet mixes resulting in "water gain" are responsible for the disintegration. In fact, it is quite probable that some effect of "water gain" is responsible for the conditions in this pier. As pointed out previously it often happens that several factors are responsible for conditions observed in structures and, that in many cases, poor aggregates are well preserved which in either more exposed conditions, or more permeable or porous concrete would have shown early signs of deterioration.

A condition which makes it difficult to determine how much of the disintegration in the pier in Fig. 9 is due to water gain is that the disintegration of the aggregate itself proceeds more rapidly where the contact with water is most frequent. Thus, the rain falling on top of the pier starts disintegration at this point. As the expansion and disruption of the aggregate proceeds the concrete becomes more porous and the water finds its way through this porous mass to the layers below. The process is continuous from the top down in exactly the same manner as that resulting entirely from water gain.

BONDING SUCCESSIVE LAYERS

Fig. 10 presents a characteristic defect that is easily recognized. In this structure can be seen the incrustation of calcium car-



FIG. 11—STRUCTURAL DEFECT DUE TO INADEQUATE DRAINAGE

bonate which had its origin in water seeping through the joints between the successive layers. Such deposits are seen both on the intrados of the arch and on the one pier in the immediate foreground of the picture. This structure was built in 1905 and it appears from the present condition that the deposits seen in Fig. 10 have not increased materially in recent years. This means that either the joints have thoroughly sealed themselves, which is a very common condition, or that water can no longer find its way to the open seam.

Fig. 11 shows one of the arches in this structure which has

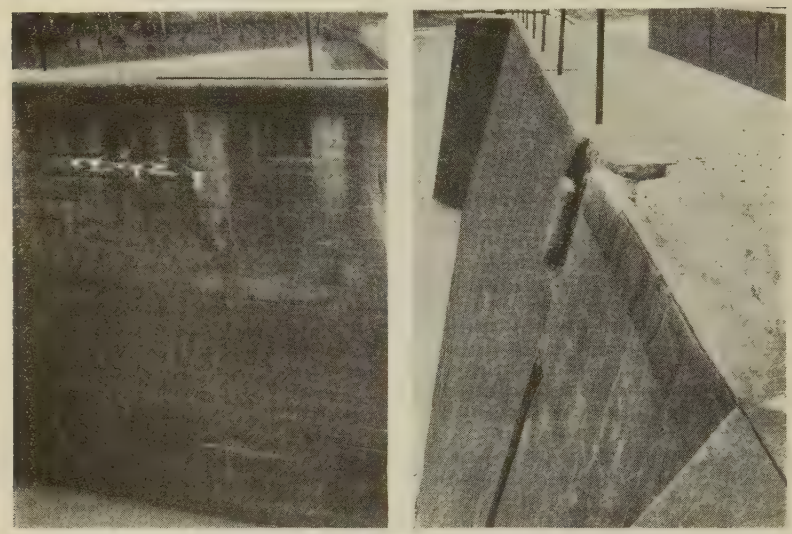


FIG. 12—A LOCKWALL WITH CORE OF LEAN CONCRETE AND FACING OF A RICHER MIX

cracked longitudinally for a portion of its length. It appears that this crack has been produced either by the hydrostatic pressure or freezing of the saturated spandrel fill. This indicates the necessity for proper drainage in this spandrel space. The fact that it has been confined to only a few of the arches in this long structure indicates that, in general, the provisions in this respect were successful. Similarly, the complete freedom from seepage at most of the other joints in the structure indicates that the methods employed for obtaining the proper bond through successive layers were generally successful.

STRUCTURES FACED WITH A RICHER MIXTURE

A condition frequently encountered is where a concrete structure is faced with a somewhat richer mixture to provide greater resistance to the weather. In some of the earlier structures this type of construction was very common. While there are successful structures of this type there are cases where the outer shell has cracked badly and separated from the interior. In such failures of this type as have been encountered, the outer shell has been relatively thin and of concrete very much richer than the inner mass.

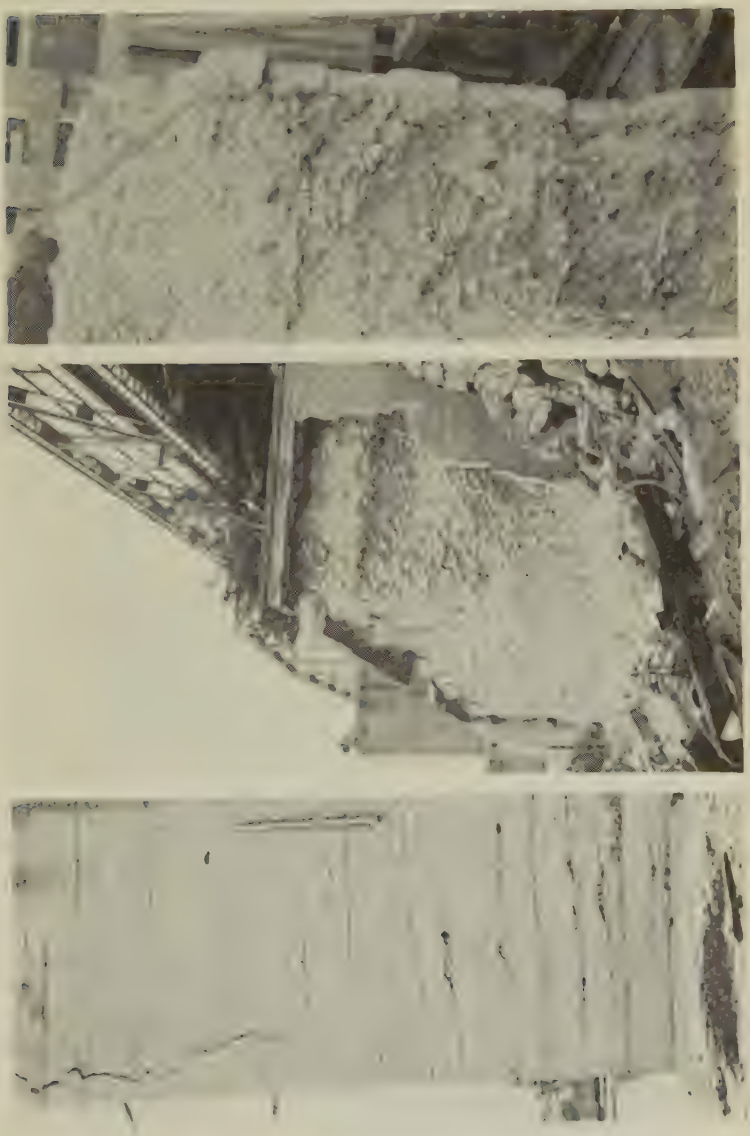
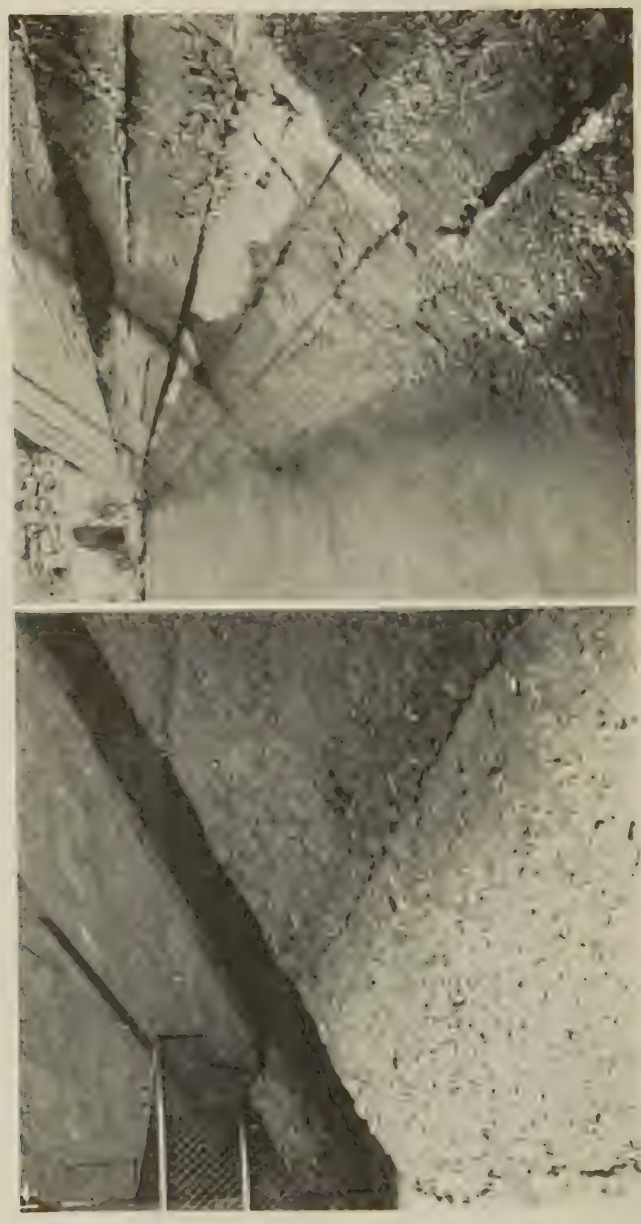


FIG. 13—EXAMPLE OF LEAN CONCRETE FACED WITH A RICHER MIXTURE



FIGS. 14 AND 15—TWO VIEWS OF A DAM OF VERY LEAN MIXTURE IN MILD CLIMATE.

Figs. 12 and 13 show two prominent examples of this type of construction. Fig. 12 is a lock wall built in 1904 and 1905. Except for the very prominent cracking which is clearly seen in the two views, the exterior concrete is in excellent condition. In the view at the right, small portions of this outer shell have been broken away. Fig. 13 shows three views of another lock wall built between 1895 and 1899. In the view at the left a portion of the outer wall is seen in which the shell is badly cracked but still in position on the wall. In the central view is seen a section of the wall where the outer shell has completely broken away and the mass concrete within badly weathered. The view at the right shows a point where a section of the wall has been cut away to provide for extensions to the structure. The extremely lean character of the mass concrete and the dense character of the outer shell is brought out very well in this view.

The feature it is particularly desired to emphasize in Fig. 13 is the character of the cracking shown in the picture at the left. The large spacing between the cracks, their large size and regularity of occurrence is typical of incipient failure in this type of construction.

QUANTITY OF CEMENT

Evidence as to the quantity of cement used in the mixture is the most difficult to obtain from an examination of the structure. An extremely lean mix gives a weak and porous concrete that is readily attacked by the weathering agencies. Such a result is also produced by inadequate curing. When evidence from sources other than the examination of the structure indicates a deficiency of cement, it is usually found that the defective concrete is more or less general over the structure unless there is some point of local attack such as the water line. This would naturally be the case also where curing had been neglected. The lean concrete in the wall at the center in Fig. 13 is a typical case of gross deficiency of cement. It is seen that this concrete was badly disintegrated over the whole surface once the protecting shell of rich concrete was removed. While curing may have been involved to some extent in this wall, the extreme leanness of the mix is very evident.

Another example of a lean mixture is shown in Fig. 14 and 15. These show portions of the downstream face of a dam in which a



FIG. 16—ANOTHER VIEW OF THE DAM IN FIG. 14

so-called sand cement was used. This cement was made by grinding portland cement with approximately equal parts of sand or crushed granite. The concrete was mixed in about the usual proportions for structures of this kind using the ground mixture as the cement portion, thus the actual cement content was about ½ bbl. per cu. yd. The views in Fig. 14 and 15 are typical of the worst portions of the downstream face. Fig. 16 shows most of the downstream side of this dam. From this it can be seen that there is some scaling here and there over the entire surface.

While the scaling due to these lean mixtures is quite evident, it is very much less than would be expected for the usual conditions. The two factors which account for the limited scaling for such lean mixes are the careful methods followed in placing the concrete and the mild climate to which it is exposed. This dam is in the arid region of the southwest where both rain and frost are rare occurrences.

A fairly complete construction report on this dam shows that every care was taken to secure proper bonding of new and old work and specially to compact the concrete between the plum stones and against the forms. One inspector was employed with



FIG. 17—BUTTRESS WALL BUILT IN 1908, SHOWING EFFECT OF "WATER-GAIN"

each of the placing gangs. Concrete was delivered in bottom dump buckets and plum stones were deposited immediately upon the freshly dumped concrete. This report shows also that some attention was given to the curing. The concrete was kept wet for a period of two weeks, which was none too much for exposure to the hot dry air of this region.

The success of this careful placing is shown by the fact that there has been practically no seepage into the inspection gallery, nor along the construction joints or fill planes. The complete absence of seepage on the downstream face, as shown by the photograph in Fig. 16 is a rare condition for structures of that period (1913-1916).

CONSISTENCY AND PLACING

Defects due to careless or wrong methods of placing and over-wet mixtures are the most common to be found in concrete structures. As suggested in a previous paragraph, other causes

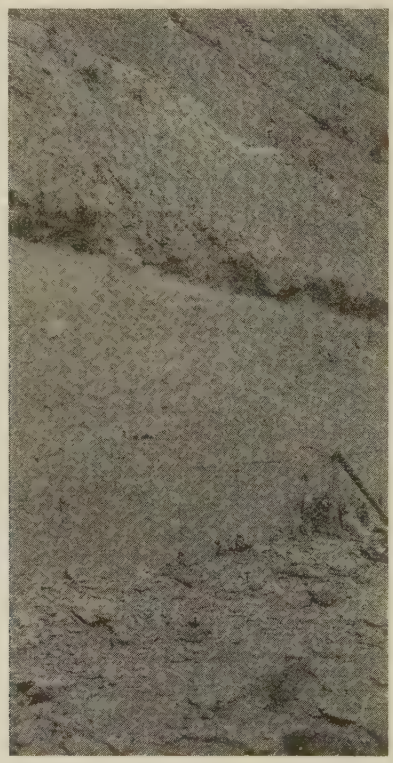


FIG. 18—PORTION OF A DAM IN SEVERE CLIMATE, ILLUSTRATING POOR CONSTRUCTION METHODS

of defects may be completely obscured by sloppy mixtures or improper placing. Evidences of "water-gain," segregation, and honeycombing are everywhere to be seen and easily identified. Closely allied with these are the cases where successive layers of concrete fail to bond properly. Sometimes these defects occur in structures where the mixtures are suitable and placing methods are otherwise satisfactory. More commonly, however, they are associated with a general carelessness in other respects. A few illustrations will suffice.

Fig. 17 shows a very common case. The stepped buttress wall, which is part of a dam in Michigan, built in 1908, provides a good



FIG. 19—ANOTHER PORTION OF THE DAM SHOWN IN FIG. 18

opportunity for water and fine material to collect during placing. The usual method is to deposit concrete in the body of the structure which allows the lighter portions of the mix to flow outward to the more distant points. The ease with which the rain and snow can collect on the steps hastens the disintegration through frost attacking the porous concrete.

Fig. 18, which shows portions of a dam in a very severe climate, is another illustration of poor construction methods. Improper mixes and segregation, due to excess water were everywhere evident on this structure. Curing also may have been neglected. The slope of the inclined fill plane to be seen just above the handle of the pick at the extreme right of the picture shows a somewhat stiffer consistency than was evident in other parts of the structure. This concrete of stiffer consistency shows much better resistance than the wetter layer just above. Fig. 19 shows another portion of this dam. In this view also the concrete of stiff consistency shown immediately above the badly honeycombed area, is re-



Fig. 20—Two-ft. laitance bed resulting from placing concrete through water



Fig. 21—Dam in northern climate. Note good condition of spillway in contrast with abutment wall



FIG. 22—CRYSTAL SPRINGS DAM, CAL. A DRY TAMPED CONCRETE PLACED IN 1886-1888

sisting the weather much better than the layer immediately below. This lower layer is almost devoid of coarse aggregate for a considerable depth below the top. This dam was about 13 years old when this picture was taken.

Fig. 20 shows a 2-ft. bed of laitance resulting from placing concrete through 14 ft. of water. Fig. 21 shows a dam in a severe climate in which the concrete in the spillway was placed with extreme care, while in the abutment there is evidence of water gain and laitance with consequent seepage at the fill planes.

These illustrations could be multiplied indefinitely with each case showing some major or minor defect resulting from the manner in which the work was carried out. It is fitting to close this discussion with an example of concrete which is well preserved because it was well placed. Crystal Springs Dam, shown in Fig. 22, is such a structure. This dam was built between 1886 and 1888 by Herman Schussler, chief engineer of the Spring Valley Water Co., of San Francisco. It is a masterpiece of good construction using the dry tamped method. Every detail of preparing the materials, measuring the quantities (including the water), mixing, placing, ramming, and curing were carried out in accordance with carefully prepared instructions. It stands as a monument to Mr. Schussler and his careful methods of construction.

Readers are referred to the next paper and to discussion of both which follow it and are referred further to the JOURNAL *for Nov., 1931, (Vol. 28) for discussion which may develop. Such discussion should reach the Secretary by Oct. 1, 1931.*

More Lessons from Concrete Structures in Service*

BY RODERICK B. YOUNG†

Any public utility, railroad or government has such a considerable investment in concrete structures that their behavior in service is a question about which they must needs be concerned. Realizing this, the Hydro Electric Power Commission decided, some years ago, to systematically study the behavior of their many concrete structures, with a view to learning not only their existing condition, but their probable condition in years to come. The examinations thus made disclosed so many interesting points in connection with the behavior of concrete in service, that it was extended to include other concrete structures in different parts of Canada and the United States, until the number of inspections now made have reached a total of several hundred. This paper is an attempt to set down briefly some of the conclusions that have been reached as a result of these examinations.

Very few concrete structures are entirely free from disintegration of some kind or other, and their defects are usually so much more conspicuous than serious that they lead the casual observer to condemn concrete as a building material, forthwith and forever. Even the critical observer is apt to become despondent by the seeming prevalence of defective concrete, and some of our well-known engineers are still in this mental condition as a result of their examinations.

However, this is not a fair view to take of the behavior of concrete in service. While defective places can be found in most concrete structures, they are usually of very limited extent

*Presented at the 27th Annual Convention, A. C. I., Milwaukee, Feb. 24-26, 1931 as part of the work of Committee 801, Durability of Concrete, see p. 1037.

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FIG. 1—LIFT LOCK AT PETERBOROUGH, ONTARIO, 26,000 CU. YD. OF ALMOST PERFECT CONCRETE WHICH HAS BEEN IN SERVICE SINCE 1904

and of minor importance. After any serious and wide-spread examination of concrete structures, one can hardly come to any other conclusion than that the greater part of the concrete in service today is in good shape and will remain so for many years to come.

One frequently finds old concrete structures in most excellent condition, resisting the destructive forces of nature, as well or better than any other material that could have been used. If anyone doubts the truth of this statement let him visit the Trenton Dam near Rome, N. Y. or the Hinckley Dam nearby, the Lift Locks at Peterborough, Ontario, the Crystal Lake Dam near San Francisco, Calif.[1] or the Thomson[2] and Pine River dams in Northern Minnesota, for these structures offer convincing proof that concrete when properly made is a durable material.

On the other hand, one occasionally meets with a structure in which the greater part of the concrete is disintegrating, but even in these cases it is the rare exception to find one so poorly made that there is not a considerable part of it in first-class shape. Even such a horrible case as that shown in Fig. 3 has its good

[1]References are listed at the end of this paper.—EDITOR



FIG. 2.—SIDE WALL OF PETERBOROUGH LIFT LOCK, WHICH IS UNDER WATER PRESSURE. NOTE ALMOST PERFECT CONDITION OF CONCRETE AFTER 27 YEARS' EXPOSURE TO SEVERE NORTHERN CLIMATE

points, and I think I can safely say, after the examination of a great many concrete structures, that not one per cent can really be classed as failures in the sense that the concrete has deteriorated or will in the near future, deteriorate to such an extent that early replacement will be required.

In any extended examination of deteriorated concrete the observer is struck with the fact that there is a marked similarity in the appearance of many of the affected areas and in the manner

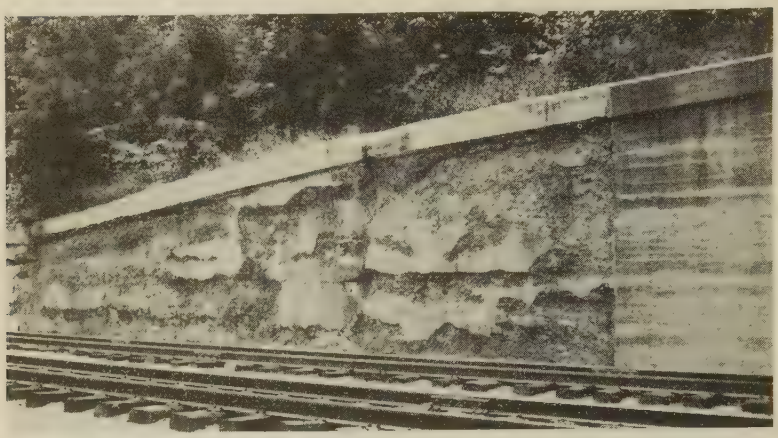


FIG. 3—EVEN SUCH A HORRIBLE EXAMPLE OF POOR CONCRETE AS THIS HAS AREAS WHICH ARE IN FIRST-CLASS CONDITION. THIS RETAINING WALL IS APPROXIMATELY 20 YEARS OLD

of their occurrence. One comes gradually to classifying them by type, and in the majority of cases finds, from the evidence offered by the concrete itself, that the circumstances surrounding the cases of a common type are similar.

By far the greater number of cases of deterioration are due to the segregation of the concrete mixture before or during its deposition. The author, in a previous paper[3] before the Institute, gave numerous examples of typical defects due to this cause and explained the manner in which they occurred. Briefly, it was shown there that the principal causes of segregation are the use of over-wet mixtures, improper methods of handling and placing, and over-manipulation of the deposited concrete.

The evils of excess water manifest themselves in water-gain and the formation of laitance at the top of "lifts" and in the accumulation of mortar and fine material in corners, at vertical joints, and under sloping forms. Improper handling and placing cause, not only the familiar honeycombed areas of gross segregation, but the even more dangerous sections of concrete that are not noticeably segregated but are sufficiently so as to be porous and permeable. Over-manipulation is frequently the result of an



FIG. 4—DEFECTIVE HORIZONTAL JOINT CAUSED BY CARELESS CONSTRUCTION. CORROSION OF THE CONCRETE ADJACENT TO THE JOINT HAS NECESSITATED REPAIRS

attempt to obtain good surfaces with poorly proportioned and harsh-working concrete mixtures, by excessive puddling and spading of the surfaces. But whatever the cause of segregation, the result is failure of the affected concrete, if the exposure to which it is subsequently subjected is sufficiently severe.

If segregation is also the most common cause of deterioration, it is also the one for which there is the least excuse, and it may be well to point out here, as will be done later, that segregation is almost entirely preventable by a little ordinary care and forethought.



FIG. 5—DETERIORATION ADJACENT TO A VERTICAL JOINT. A POROUS CONCRETE MADE FROM PIT-RUN GRAVEL PLACED WET

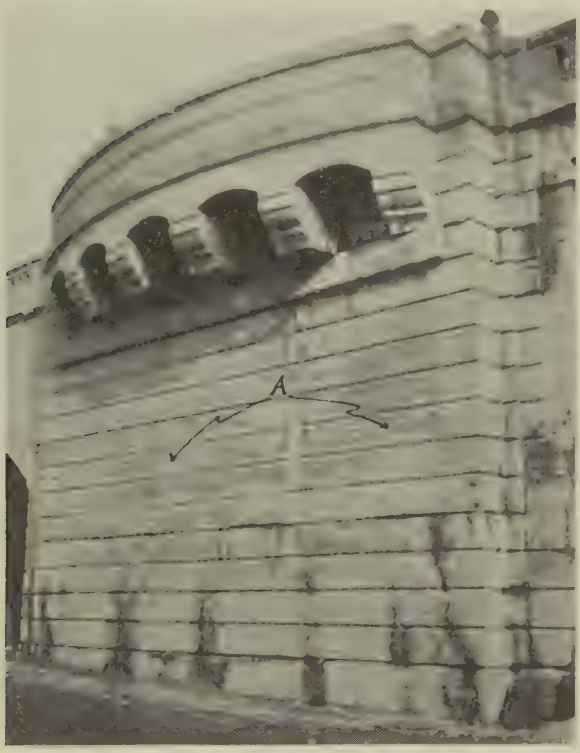


FIG. 6—POOR DRAINAGE NECESSITATED THE REPAIR OF THIS BRIDGE PIER AT "A." NOTE THE DETERIORATION OCCURRING ADJACENT TO LIME ENCRUSTATIONS

After segregation, probably the next most common type of deterioration is caused by joints, either vertical or horizontal, which are not water-tight. With these might also be classed cracks due to shrinkage or structural movements, through which water seeps. Where the concrete adjacent to the joint or crack is of poor quality, due to segregation or other causes, the deterioration may become serious. Where the concrete is good, the effect of the presence of water is likely to cause only an unsightly blemish and so the potential danger of the situation is overlooked. That there is a very real danger in the continued passage of water

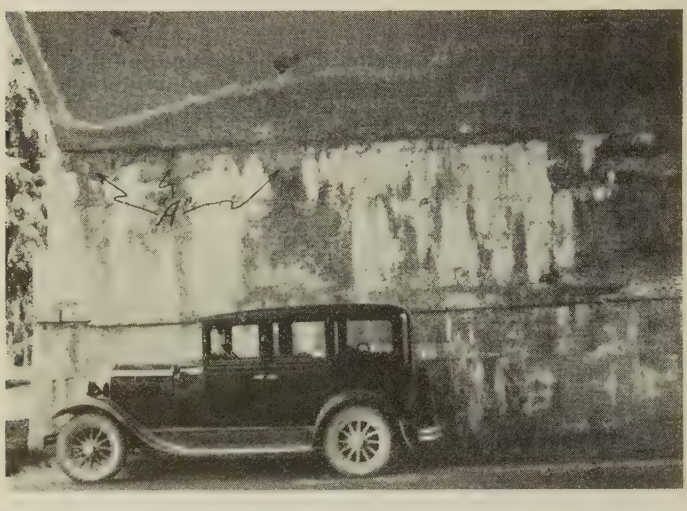


FIG. 7—ANOTHER EXAMPLE OF THE DETERIORATION OF A BRIDGE PIER CAUSED BY LACK OF DRAINAGE. NOTE AGAIN THE PRESENCE OF LIME ENCRUSTATIONS AND THE PATCHING AT "A"

through concrete is evident if one realizes that practically all its cementitious material is water-soluble and may be removed by solution if enough water is available.[4] The presence of calcium carbonate deposits, sometimes in great quantity, wherever seepage occurs is proof, if other proof is needed, of this statement.

Trouble of this kind is particularly likely to occur in hydraulic structures such as dams and conduits, but is also commonly found is retaining walls and bridges. In the majority of hydraulic structures there is no other corrective except to make the joints water-tight, which can be done by exercising the greatest care in their construction, and by the use of waterstops where, for any reason, movement of the joint may occur. In the case of retaining walls and bridges, much may be done by water-proofing and drainage. Engineers designing reinforced concrete bridges should give a great deal of thought to these points for one finds many very fine structures which would not be in their present unsightly condition if they had been properly drained.

Another very frequent cause of deterioration, especially along the sea coasts, is the corrosion of the embedded reinforcing steel.

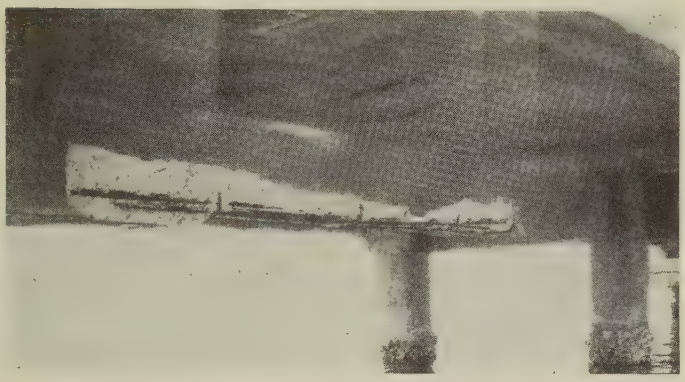


FIG. 8—ONLY 10 YEARS OLD, BUT THIS BRIDGE HAS HAD TO BE ABANDONED DUE TO THE PREVALENCE OF SIMILAR CONDITIONS

The oxides of iron, which form in the presence of moisture and oxygen from the corrosion of steel, occupy roughly two to three times the space of the original steel they replace, and consequently a reinforcing bar, if allowed to rust, will crack or spall the concrete surrounding it.

This type of deterioration is very prevalent along the sea coasts, although it is by no means confined to those localities. In the southern parts of the continent where frost action is absent, it is by far the most serious form of deterioration and the most common. It is serious because in many cases any considerable spalling of the concrete endangers the safety of the structure and may even cause its failure.

Since there seems to be no practical method of rust-proofing the reinforcement, the obvious remedy is to embed the bars in dense impermeable concrete to a depth sufficient to protect them from moisture and air. Seemingly this should not be difficult to do, as one can discover instances where as little as one-half inch of concrete or mortar gave complete protection to the steel. The present excellent condition of most of the concrete ships built during the war is proof of this, and other examples such as that illustrated in Fig. 9 are not uncommon.

However, even if the engineer designs his structure to provide sufficient embedment of the reinforcement and specifies and



FIG. 9—ONE-HALF-INCH OF DENSE, IMPERMEABLE CONCRETE HAS PROTECTED THIS STEEL FOR SIX YEARS

obtains a dense durable concrete with which to cover it, he has not done his complete duty until he designs his reinforced concrete members in such a way that the quality concrete that he has provided can be placed properly around the reinforcing steel. If the bars are so concentrated and spaced that it is impossible to get concrete around and through them without the use of excess water or over-manipulation, one can hardly blame the concrete or the workmanship for any deterioration that subsequently occurs. In the case illustrated in Fig. 9, a covering of one-half inch of the concrete had protected the steel for six years. The concrete here had the hardness and appearance of one that would test 3500 lb. per sq. in. or better, and apparently had been well cured. However, the beams made of this same concrete repeatedly show cracking along the line of the tension reinforcement, and as far as one could determine, the probable cause was the difficulty of properly placing the concrete in the bottom of the beam below and alongside of the steel.

Certain localities are blessed with very fine aggregates and in these, deterioration of the concrete due to unsound aggregate is practically non-existent. In other localities, not so blessed, cases of deterioration due to the failure of the aggregate are fairly common. The author has had the opportunity to observe con-

crete structures in several of both types of localities and while there is no question but that the quality of the aggregates has an important influence on the condition of the structures, unsoundness of the aggregate alone is seldom the primary cause of disintegration, although it is usually a contributory factor where other detrimental conditions exist.

This statement is probably more nearly true for the coarse aggregate than for the fine. In most of the cases that the author has examined where the coarse aggregate was unsound, it seemed probable that if the mortar surrounding the coarse aggregate had been of better quality, the unsound aggregate alone would not have been able to cause disintegration of the concrete. It is not at all uncommon to see evidence, such as in Fig. 13, where a great deal of unsound coarse aggregates has been used and still the concrete was generally in good condition. On the other hand, where unsound coarse aggregate is to blame for the disintegration of the concrete, the failure is usually distinctive and complete.

Unsound coarse aggregate may be protected by being surrounded by a durable mortar, but if the mortar itself is made up of unsound fine aggregate, the concrete is doomed. In such cases, the failure is a true disintegration and takes place relatively rapidly, usually manifesting itself within a very few years by a general deterioration and softening of the concrete.

There is a belief, in some quarters, that unsound cement is the cause of most of the deterioration of concrete, while others argue to the contrary. One occasionally meets with a clear-cut case of disintegration where all the evidence points to the cement as the cause, but these cases are rare. The author has made a careful examination of some dozens of deteriorated structures, the condition of which was blamed on the cement, and found therein so much evidence that other factors such as those already discussed, were mainly responsible for the present condition of these structures, that he has been forced to the conclusion, that whereas there may be cements which, although passing all the usual recognized tests, are still unstable in concrete, yet they do not have any important deleterious effect until other improper conditions are permitted to exist. The reasonable conclusion is, therefore, that if a cement meeting the present accepted standard



FIG. 10—BOW OF THE CONCRETE SHIP "FAITH" BUILT IN 1918.
NO DETERIORATION HERE

specifications is used in a structure, future deterioration caused by the cement alone is extremely unlikely.

Any discussion of the behavior of concrete in service is hardly complete without reference to the agencies that bring about their deterioration. It should always be remembered that concrete structures are exposed to the same weathering conditions which, through the ages, have levelled whole mountain ranges and totally changed the aspect of continents. It should also be remembered that the same weathering agencies are proving destructive to stone masonry and to brickwork, and that only the complete protection of iron and steel structures save them from being

destroyed in a very short time. The following little couplet appearing in the *Engineering-News Record*[5] expresses this thought os aptly as to be worth repeating:

> "Bricks will break and rocks will fall,
> Paint will peel, and concrete spall,
> Wood will rot, and steel will rust,
> Ashes to ashes, and dust to dust."

Concrete has many enemies in nature, but by far the most important is the combined action of water and frost. One has only to make a tour from north to south, examining concrete along the way, to discover the part that frost plays in the deterioration of concrete. Below the frost belt, even the poorest concrete will give service for years without appreciable deterioration, and only when used in a reinforced concrete structure where corrosion of the embedded steel causes spalling, will its poor quality become conspicuous. One finds many structures in Florida and along the Gulf of Mexico that are ten or more years old and in first-class condition, which if north of the Ohio River would, in the same period, have required extensive repairs or replacement.

It should be remembered in dealing with the action of frost that it is not the freezing of the concrete itself that is harmful, but the freezing of water that is present in the concrete. Dry concrete, no matter what its quality, is not destroyed by alternate freezing and thawing, but let the pores of that concrete be filled with water to such an extent that no space is available to take care of the expansion that inevitably takes place when water changes to ice, and disruption must occur unless the concrete is strong enough to resist the large stresses set up. If one considers that concrete is relatively weak in tension, one sees the reason why, in making concrete resistant to frost action, impermeability is so much more important than strength.

It should also be remembered that while concrete may resist one freezing or several when saturated with water, yet the continued repetition of this action will in time destroy it. Further, disintegration due to frost proceeds from the surface inward and only a few degrees of frost are required to cause freezing of the contained moisture in the surface layer. It would seem, therefore, that the effects of frost should be more severe in those sections of the continent that have the greatest number of alternate



FIG. 11—DETAIL OF BOW OF S. S. "FAITH." NOTE FREEDOM FROM STEEL CORROSION, ALTHOUGH SOME OF THE REINFORCEMENT IS ONLY EMBEDDED ONE-HALF INCH

freezings and thawings, and while this is not subject to statistical proof, it seems to be borne out in a general way by observation. It certainly is a fact that as soon as one enters those sections having more than a very few annual frosts, the evidence of frost action is almost as common and as conspicuous as in districts further north, so that it behooves the engineer in those sections having mild winters to remember that his concrete structures are subject to as severe, and possibly more severe exposure than those in more rigorous climates.

If frost is not a destructive agent except in the presence of water, then it might truthfully be said that water is concrete's worst enemy. As already mentioned, water alone will destroy concrete through its solvent action, but more important, water is the medium through which most of the other enemies of concrete carry on their attack. Sea water, sulphate-bearing waters, and bog waters are merely waters carrying impurities that render their destructive action more severe, and all act through the intimate contact that the water has with the concrete through absorption in its pores.

Perhaps the author is inclined to over-rate the importance of water as a destructive agent due to his long contact with hydraulic structures, but it is also possible that others, differently situated and not having had the same chance to observe the



FIG. 12—UNSOUND AGGREGATE CAUSED THE CONDITION OF THE CONCRETE AT THE RIGHT. THE CONCRETE AT THE LEFT IS SIMILAR IN ALL RESPECTS, EXCEPT THAT A DIFFERENT COARSE AGGREGATE WAS USED

destructive effects of water, under-rate its possibilities instead.

Research tells us that, given the opportunity, any water will, in time, completely remove the binding material which is formed in concrete by the hydration of cement[4,6,7] and a few cases of the kind have been reported in the technical press[6,8,9,10,11]. However, for the most part, engineers are inclined to think of such action as a remote possibility, but in this, they are wrong, for it is a quite common occurrence around hydraulic structures, although its true nature is often obscured by being combined with frost action.

Mention has already been made of the solvent action of water adjacent to leaking joints or cracks, but it is by no means confined to these places. Porous concrete, where totally submerged is water, will have the cementitious material dissolved therefrom in a few years, even when the water in contact with it is hard. The author has had three cases of this kind requiring major repairs, occur in his work in as many years, besides two others where the solvent action of water was the primary destructive agent, but was assisted by the surface action of frost.



FIG. 13—SPALLING DUE TO UNSOUND COARSE AGGREGATE. THE EFFECT OF THE UNSOUND AGGREGATE IS CONFINED TO THE SURFACES

Another manifestation of the solvent action of water is regularly found wherever concrete has been in contact with soft pure waters for any length of time. In such cases, the water dissolves the surface coating of cement mortar, exposing the underlying aggregate. This action takes place relatively rapidly at first, until the aggregate has been exposed, after which the action becomes very slow for then the only cement in contact with the water is that between the aggregates. The author has examined concrete structures less than two years old that have been etched in this way, and others from ten to thirty years old that had a deeper etch, but still no unravelling of the affected surfaces. In hard water this etching action is very slow.

Examples of the solvent action of water may often be found where drainage conditions are such that rain water runs over the surface of concrete. The same phenomenon will be found also on the tops of cornices, handrails, sills, and the treads of steps, and along the lower edges of the same cornices, handrails and sills, where water collects during rains.

None of these surface manifestations of the corrosive action of water are serious, but give a quantity of that same water a

chance to penetrate the concrete and the results may be very dangerous.

A study of the characteristics of those waters that have proved most corrosive to concrete, reveals the interesting fact that, whereas the hydration products of cement are soluble in any water, those that are most destructive are relatively pure waters carrying so little CaO in solution that they are lime hungry and having an acid reaction as indicated by a hydrogen-ion concentration below 7.00. These observations have been confirmed by investigations recently reported in Germany.[12]

The addition of certain salts to a water will increase its destructive properties. Sea water and the sulphate-bearing waters of the west are both cases in point, and in both, the salts involved are principally the sulphates of magnesium, sodium and calcium. The action of these compounds on the concrete appears to be both chemical and mechanical, the chemical being partly a corrosive action and partly an interference with the normal processes of hydration,[6,13] the mechanical due to the disruptive effects of the by-products of the chemical reactions, which forming compounds within the cementing matrix of the concrete require more space than the materials they replace.

A great many natural waters, both ground and river, contain one or all of the sulphate salts in solution, but unless the quantity is considerable, their presence is unimportant. However, in some parts of both western Canada and the United States, there are localities where the ground waters contain dangerous concentrations of these sulphate salts and where sulphate action becomes a matter of major importance to engineers, but generally speaking, sulphate determination is a local, rather than a continent-wide problem. On the other hand, it is found elsewhere occasionally and the engineer asked to diagnose a case of disintegrated concrete and prescribe for its treatment should not overlook that fact. Its manifestations are usually, but not always, distinctive and may easily be obscured by frost action. Where there is any suspicion of sulphate action, a chemical analysis of the disintegrated concrete will show whether or not one's suspicions are justified.[14]

All sea water structures are liable to sulphate action due to the presence of these salts in the ocean. However, while the



FIG. 14—A POROUS CONCRETE 20 YEARS OLD WHICH WOULD HAVE RAPIDLY DISINTEGRATED IF SUBJECTED TO FROST. MADE OF FINELY-GRADED SEA SAND AND SHELLS

author's observations of sea water structures have been limited, it has led him to question whether the destructive tendencies of sea water were due to the salts it contains as much as to the fact that it was water. One can duplicate in the harbors of the Great Lakes all of the types of deterioration that one sees on the North Atlantic seaboard. Along the southern coasts below the range of frost, there is little evidence of chemical deterioration of the concrete, other than the surface etching met with in fresh water where it is of a corrosive type. There is a great deal of spalling due to corrosion of the embedded steel, and while this type of action will naturally be more severe in the presence of salty moist air, it is by no means confined to the seacoast, but is quite common elsewhere. There is the further fact that sea water does not contain high concentrations of the sulphate salts. The author, therefore, is inclined to believe that most of the deterioration of concrete in sea water is due to the same factors which cause it elsewhere and not, as usually claimed, to the presence of sulphates.

We have now considered both the effects brought about by the deterioration of concrete and the principal agencies that have acted to bring these effects into being. We have also considered,



FIG. 15—ANOTHER SOUTHERN WALL OF VERY POOR CONCRETE, WHICH EXCEPT FOR LOCAL SPALLING DUE TO AGGREGATE, IS IN FAIR SHAPE

though rather briefly, some of the faults of workmanship that usually accompany deterioration. Can we not now deduce from the information thus obtained, what property of the concrete renders it susceptible to attack?

The answer to this is neither difficult nor new; the author pointed it out some years back,[15] and others have done likewise. The fundamental condition which is common to almost every disintegrated concrete is that it is porous in the sense that it is absorptive or permeable to water. Prevent the penetration of water into concrete and at once it becomes a building material highly resistant to all ordinary destructive agencies. Even in the case of those waters which will destroy any concrete in time, such as concentrated sulphate waters, the rate of deterioration is greatly reduced where it is confined entirely to the exposed surfaces, as it must be if the concrete is impermeable.

If one accepts this conclusion, as one almost must after a critical review of the evidence offered by concrete everywhere, then the problem of durable concrete becomes the problem of making it dense and non-porous, or speaking in terms of measureable properties, impermeable and non-absorptive. Reduced to these terms, the problem is theoretically easy of solution for the fundamental factors in the making of a dense, water-tight, high-

strength concrete have long been known and are generally well understood. Practically, the problem is more difficult, for the degree of skill and care used in handling and placing concrete is of paramount importance in making it durable, and proper workmanship in these particulars is not always easy to obtain.

No attempt will be made here to consider the factors necessary to produce a dense, impermeable and strong concrete. If the reader is not already familiar with them he has only to consult the recent technical literature[16,17] to find them discussed in full, but it might be useful for the author to consider briefly here a few points that his observations have led him to believe are important in any concrete which is to be subjected to severe exposures.

One of the conditions that frequently give difficulty is the use of harsh, unworkable mixtures. A concrete mixture deficient in mortar is not only more likely to segregate in handling, but even if this is prevented, it requires a great deal of spading to give presentable surfaces, which, in turn, causes scaling. Mixtures of this type are also inherently porous, because their harshness is usually due to a lack of mortar or to a mortar which is itself harsh from a deficiency in the grading of the fine aggregate. In addition to these drawbacks, a harsh mixture is more apt to segregate in handling, and because of its lack of workability, there is much more chance that some well-intentioned individual will attempt to overcome its deficiencies by the addition of water.

Another frequent cause of disintegration is the use of over-lean mixtures. Cement plays a dual role in concrete; in combination with water, it forms the glue that binds the aggregates together, but it and its products of hydration also fill and seal the smaller spaces in the aggregate mass. Any reduction in cement content beyond a certain amount is accompanied by an increase in porosity until a point is reached where, if the exposure is severe, disintegration is almost inevitable.

Instances of disintegration due to lack of cement are by no means uncommon and several have been reported to the Institute. The matter is one that every engineer would do well to remember when he specifies concrete in terms of its compressive strength. With the modern cements, it is now taking less cement to obtain concrete of a given 28-day compressive strength than it did eight or ten years ago, and the result is that quality as measured by



FIG. 16—POCKETS OF SEGREGATED CONCRETE FROM WHICH CEMENT WAS TOTALLY REMOVED BY SOLUTION. NOTE POSITION OF WATER LINE AT "A" AND SLIGHT ETCHING OF SURFACE, WHERE WATER IN CONTACT WITH IT IS HARD. COMPARE WITH FIG. 18

strength is not the same today as then. As an example, take the experience of the Hydro Electric Power Commission of Ontario. In 1920 the Commission built a hydro-electric plant and used approximately seven sacks of cement per cu. yd. to produce a concrete testing 2500 lb. per sq. in. at 28 days. In 1930 they built another plant a few miles away on the same river with aggregate and cement from the same source and using five and three-quarter sacks per cu. yd. obtained concrete testing 3500 lb. per sq. in. at 28 days. Concrete of the same strength as that made in 1920 could have been produced with about four and one-quarter sacks per cu. yd., but the latter, while as strong, would not have been as impermeable, and hence as durable and so in this case the governing feature in the choice of the cement content was durability and not strength.

Strength can never safely be taken as a measure of impermeability in a concrete, for while it is true that for a given set of conditions, the greater the strength of a concrete the less its permeability, it is not true that a certain degree of permeability will exist in all concretes of the same strength regardless of the



FIG. 17—THE FIRST CONCRETE PUT IN UNDER WATER-CEMENT RATIO CONTROL, BUILT IN 1919

aggregate, the cement content, or the treatment that it has received. A concrete of today is probably more permeable and hence less durable than concrete of equal strength made ten years ago, and yesterday's standards of quality no longer apply to today's concrete. It is the author's opinion that the widespread adoption of the strength specification for concrete is creating a situation dangerous to the industry, for the same engineers who are now adopting the newer forms of specification are still using the old standards of strength as a measure of quality, in the belief that in meeting these they are assured of a durable weather-resistant concrete.

Segregation of every form is extremely dangerous to the permanence of concrete. Some of the consequences of segregation have already been mentioned in the beginning of this paper and the practices which most frequently cause segregation were touched upon. But, whether or not segregation is the result of excess water, improper handling, a poorly designed mixture, or over-manipulation, the fact remains that practically all segregation is easily prevented by the exercise of ordinary care and forethought, and there is no excuse for any architect, engineer or contractor to tolerate its presence in any concrete work for which he is responsible. He has a right to demand and expect

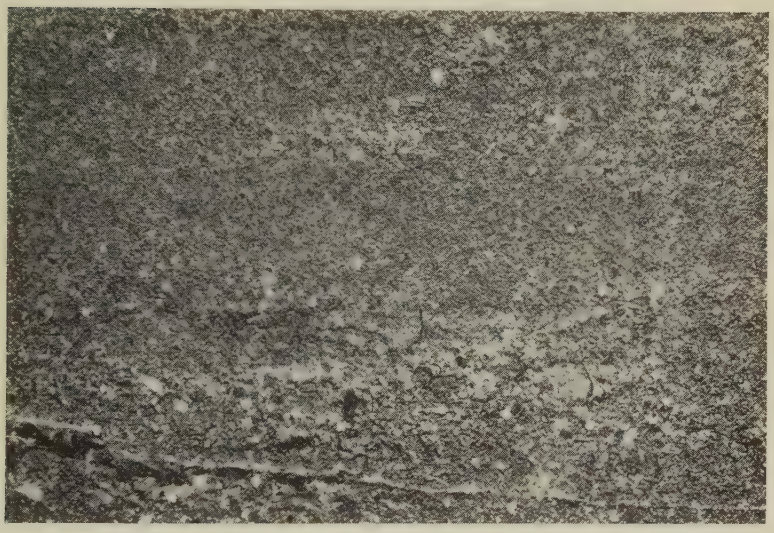


FIG. 18—CLOSE-UP OF A PART OF THE CONCRETE SHOWN IN FIG. 17 WHICH HAS BEEN ETCHED BY BEING IN CONTACT WITH A CORROSIVE WATER

good workmanship in these particulars for it costs no more to handle and place concrete without segregation than with, particularly if the effect of segregation on the cost of finishing is taken into account.

Many a concrete job falls short of perfection because of lack of the proper kind of supervision. There has to be on the job some individual with a sound knowledge of concrete armed with the proper authority, whose duty it is to see that the quality of the materials, standards of workmanship, and the after-treatment of the concrete are as they should be. The success of those outstanding cases of durable concrete mentioned in an earlier paragraph was due in no small degree to the fact that such a man was connected with the job. The Petersborough Lift Locks had Walter J. Francis; the Crystal Lake Dam, Herman Schussler; the Thomson Dam, H. C. Ash, and so on, and they are enduring monuments to these men.

Supervision of a concrete job has to be intimate and continuous to be successful. It is not enough to make a daily visit to a job as is the too common practice of many architects and engineers.

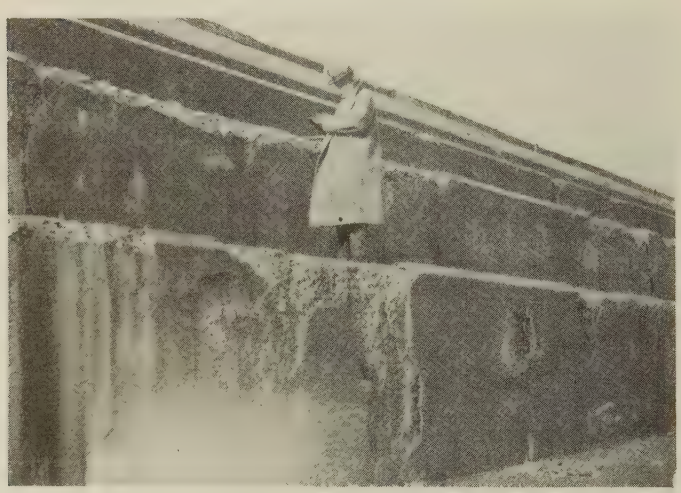


FIG. 19—SCALING AND DISINTEGRATION DUE TO HARSH WORKING CONCRETE MIXTURES AND THE CONDITIONS ARISING THEREFROM

Someone should be present immediately before and during the placing of any important concrete. Such supervision adds but little to the cost of a structure and is cheap insurance to pay for a job that will be free from future trouble. And it is the common experience of those dealing with competent inspectors that in many indirect ways they effect economies of operation that more than offset the cost of their services.

Summing up, the author has arrived at the following conclusions as a result of his experience with concrete good, bad and indifferent:

(1) Generally speaking, concrete structures are satisfactorily performing the functions for which they were built.

(2) While few concrete structures are wholly free from deterioration, the excellent condition of the greater part of even those which would be classed as examples of poor concrete, and the outstandingly good condition of others 20 or more years old, are proof that concrete is and can be made durable.

(3) Most cases of deterioration can be classified into a few types to which definite causes can be assigned.

(4) Most of the defective concrete is due to faults of workman-

ship, the use of excess water, and other causes that are strictly preventable by the exercise of reasonable care and supervision during construction.

(5) Faulty construction joints are very prevalent and under certain circumstances may start dangerous deterioration.

(6) Unsound aggregate can cause serious disintegration, but trouble of this kind is not general and usually does not become a factor in destroying concrete unless the concrete is otherwise unsatisfactory.

(7) Cement is seldom a primary cause of concrete deterioration.

(8) The principal naturally occurring destructive agents affecting concrete are frost, water, and the corrosion of embedded steel.

(9) Water, if allowed to penetrate concrete may, and often does, totally destroy it due to its solvent action on the cementitous binder.

(10) Fundamentally, most concrete deterioration is due to its being porous.

(11) The best recipe for durable concrete is to make it so dense and impermeable that water cannot enter.

(12) Harsh, unworkable concretes are prone to deteriorate.

(13) Lean concretes, no matter how strong, are not durable.

(14) Intelligent and constant supervision of concrete during construction is necessary to insure durable concrete.

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DISCUSSION BY L. W. WALTER*

IN THE papers presented by Mr. McMillan and Mr. Young, the authors have brought to us some impressive lessons arising out of their experience in work directed along the lines of the activities of Committee 801. Coming as they do from men who have made it their business to acquaint themselves with the performance of concrete structures of various types under different conditions of exposure, and whose business it is to know how to overcome the deficiencies which they have observed, and which their investigations have revealed, these papers commend themselves to the careful study of all those who have been less favored by opportunity and experience.

It is brought out that, of the factors affecting the durability of concrete, the one of most concern is the action of frost on saturated concrete. It follows from this that the most important requirement of concrete, in the interest of durability, is watertightness. This requirement is essential, not only in freezing climates, but in concrete exposed to corrosive waters, and in reinforced concrete structures in or very near sea-water.

It has also been my privilege to examine many structures with which I have been familiar from a standpoint of construction and conditions of exposure, and to have accompanied the authors of these papers in a survey of many structures in a climatic range extending from Key West, where frost is unknown, through the northern part of the United States and southern Canada,

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where frequent cycles of freezing and thawing have their telling effect on concrete of inferior quality, and into the north woods where a diminishing number of freezing and thawing cycles seem less severe in their toll, even though the winter temperatures are low. My conclusions from these examinations are in agreement with those of the authors of the papers presented.

There is one important matter brought out in Mr. Young's paper which I particularly desire to emphasize, namely, that strength alone is not a dependable measure of water-tightness or durability.

In the design of concrete to meet the fundamental requirements for strength, durability and economy, too little consideration is being given to the fact that, as measured by 28-day test results, present day cements carry higher strength-giving quality to the concrete than was characteristic of cements of earlier periods, but do not produce correspondingly higher ultimate water-tightness when used in like proportions.

The gradual but constant stepping up from year to year in strength-giving quality of cement, as determined by the 28-day tests, without its contributing, in a corresponding degree, to higher ultimate water-tightness of concrete, calls for extreme caution in the selection of a particular class of concrete, classified on a strength basis.

Unfortunately, many users of concrete are overlooking the basic principle of the water-cement ratio law, and, in trade practice, are depending upon strength as the sole measure of quality. With present day cements, it is possible to get relatively high 28-day strength of concrete with mixtures lean in cement and with a high water-cement ratio. Such mixtures are too porous to withstand unfavorable exposure.

In order to stop this trend in trade practice, and to safeguard against the use of these porous mixtures, it seems necessary to indicate in our specifications (separately for each class) not only the maximum allowable amount of water per bag of cement, but also the minimum allowable amount of cement per yard of concrete.

Readers are referred to the JOURNAL *for Nov. 1931, (Vol. 28) for further discussion of the two preceding papers which may develop. Such discussion should reach the Secretary by Oct. 1, 1931.*

The Permeability of Gravel Concrete*

BY PAUL T. NORTON, JR.† AND DAN H. PLETTA‡

INTRODUCTION

The research project reported in this paper was begun three years ago in the Mechanics Department at the University of Wisconsin in the hope of determining the relationship between permeability and water-cement ratio, and also the relationship between permeability and such other properties as strength, consistency, absorption, cement-voids ratio and grading of aggregate.

A number of permeability tests had previously been made in this laboratory. In none of these earlier tests was the water-cement ratio computed, although F. R. McMillan has figured the water-cement ratio for the concretes reported in University of Wisconsin Bulletin 1245 (1923) and has plotted the flow of water against these values.

The data used in establishing the results of these observations were obtained from a series of tests repeated five times in order to aid in securing accurate averages. The program included the making and testing of 34 different mixes, with each of which three or four different amounts of water were used, ranging from that required for normal consistency to 35 per cent more than that amount. The mixes were designed to cover the range from those suggested by the Fuller-Thompson curve to others which were much over-sanded. A complete series of 34 mixes was made and the series repeated five times in order to eliminate as much as possible such things as the aging of cement, the varying relative humidity, the change in temperature, the curing condition of the

*Presented by Dan H. Pletta at the 27th Annual Convention A. C. I., Milwakee, Feb. 24-26, 1931.

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At the time these tests were made Mr. Norton was Assistant Professor of Mechanics and Mr. Pletta was Instructor in Mechanics at the University of Wisconsin.

specimens, the personal equation, etc. With each permeability specimen made from a given batch, two 6 x 12-in. cylinders were molded. One of these was used to obtain the compressive strength of the concrete, and the other for absorption testing. Nearly 2000 specimens were made and tested.

This research project was made possible through the research fund of the Mechanics Department of the University of Wisconsin. The authors are grateful to the institution with which they were formerly connected for the opportunity of carrying on this work and to Prof. M. O. Withey for his careful supervision of the tests and helpful suggestions in the writing of this paper. The authors also wish to thank M. E. Basford, Azmi Jemil, Robert Greiling, Leo Janicki, M. O. Scott and the others who assisted in the tests, and also the Atlas Portland Cement Co., the Lehigh Portland Cement Co., and the Universal Portland Cement Co. for their donations of the cement.

MATERIALS

Cement—The cement used in these tests was a mixture of equal parts of Universal, Lehigh and Atlas portland cement. Twelve barrels of each were mixed and stored in bulk in a large galvanized iron bin, which was made as air tight as possible by having the seams calked with tar and the door edged with felt. About 20 lb. of each brand was removed before the mixing and used for testing the individual brands and a like amount of the mixture set aside for a similar purpose.

The results of these tests are shown in Tables 1 and 2. Table 1, showing the chemical analysis of the individual cements, represents the tests of the respective manufacturers. Table 2 outlines the physical properties of each of the cements and of their mixture as determined at the laboratories of the manufacturers and the University of Wisconsin testing laboratory. The tensile and compressive test specimens were made, stored and tested according to A. S. T. M. specifications. The loose weight per cubic foot of cement was assumed to be 94 lb.

Sand—Mixes "A" to "a" inclusive, see Table 6, were made of Janesville sand, and "b" to "k" inclusive of University sand. Janesville sand is about two-thirds quartz and one-third dolomite; University sand is almost entirely quartz. Tables 3 and 4 show

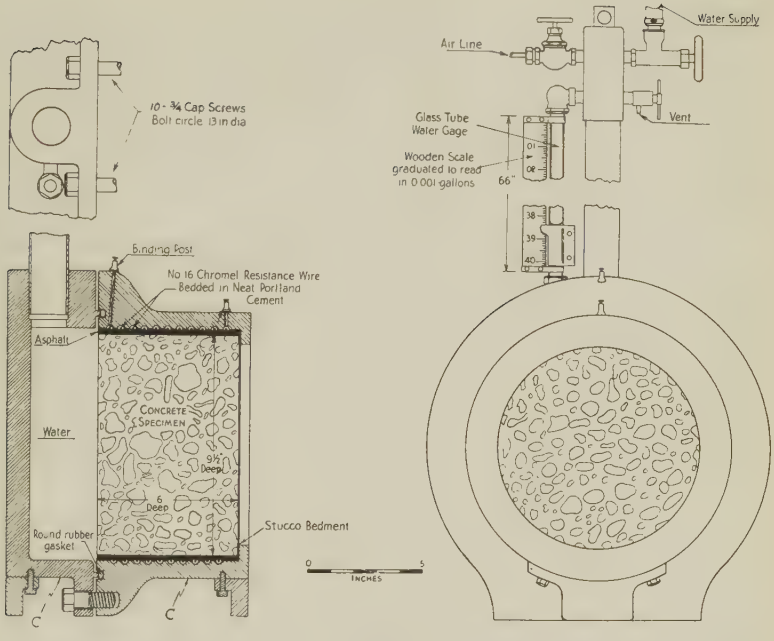


FIG. 1—APPARATUS FOR TESTING THE PERMEABILITY OF CONCRETE TO WATER

the physical properties of the two fine aggregates. All of the sand was screened through a ¼-in. sieve before it was tested or used.

Gravel—As the gravel was received it was screened into three sizes: ¼ to ⅜, ⅜ to ¾ and ¾ to 1½ in. The specific gravity and absorption of the three sizes varied somewhat, and since the individual sizes were combined to make the five different coarse aggregates used, the specific weights, specific gravities and absorptions also differed. This information is shown in Table 5. All of the material was Janesville gravel with the exception of the largest size used in the "Z" and "a" mixes. This was obtained from a gravel pit near Madison to reduce expense.

Mixed Aggregate—The sand and gravel were combined into 34 different mixes and the characteristics and proportions of materials in each are shown in Table 6.

Preparation of Materials—The sand was air dried before it was used by spreading it on the floor of the testing laboratory and

raking it occasionally for a day or two. The gravel was air dried in separate storage bins. About four hours before mixing the moisture content both of the sand and of the individual sizes of coarse aggregate to be used was determined by weighing a 500-gram sample, heating it in an oven to constant weight and then weighing it again. This information was used to determine the amount of water to be added to a particular batch so that the water-cement ratio would remain constant for individual batches throughout the series of tests. The temperature of the mixing water was kept at approximately 70 deg. F.

Weighing the Materials—The cement and aggregates for each batch were weighed on a scale sensitive to 0.05 lb. and the water on a scale sensitive to 0.01 lb. After the cement, sand and the individual sizes of coarse aggregate had been weighed separately they were weighed collectively for a check.

MAKING THE SPECIMENS

Mixing the Materials—After the materials had been weighed the sand and cement were placed on a metal mixing tray about 3 ft. 8 in. by 5 ft. 6 in., the surface of which had just been moistened. The cement and sand were first mixed to a uniform color by two men, working from opposite sides of the tray, turning the batch over twice with square pointed shovels. The coarse aggregate was then added and the mixing repeated. The water was poured into a crater formed in the aggregate and the material forming the crater was worked in toward the center. After the batch had been allowed to stand for a half minute it was mixed by turning over fourteen times.

Slump and Flow Tests—The slump and flow tests were made for each batch 10 minutes after completion of the mixing. There was a twofold purpose in the delay; it allowed the aggregate, which varied in moisture content from time to time, to absorb enough moisture so that the consistency remained almost constant for the same batches later in the series and it provided ample time to weigh materials for another batch.

Setting Up and Filling the Molds—The molds for the 6 x 12-in. cylinders used for the strength and absorption tests were set up on leveled steel plates. The molds for the permeability specimens were made of cylindrical metal sides with wooden bottoms

that had been thoroughly soaked in oil. The molds were oiled each time before they were filled.

Each batch of concrete was placed in the three molds as soon as possible after the slump and flow tests had been made. The 6 x 12-in. cylinders were filled one-third full, then tamped 25 times with a $\frac{5}{8}$-in. rod, then refilled another third, etc., and the top finally finished with a trowel. A metal plate was used as a cover to prevent evaporation. The permeability mold was filled half full and tamped 25 times, then filled completely and re-tamped, and the mold leveled off with a tamping rod. These molds were covered with wet cement sacks to retard drying.

Stripping the Molds—The molds were removed after 24 hr. The outer film of concrete on the top and bottom of the permeability specimen was removed with a wire brush and one of the other cylinders, later to be used for the absorption test, was subjected to a similar treatment on all surfaces. The other specimen was used for the strength test. The marks identifying the batches were painted directly on the specimens.

Curing the Specimens—As soon as the molds had been stripped and the specimens painted they were placed in a moist closet where they remained until 28 days old. Not more than eight batches were made at any one time, for the permeability apparatus was designed to test a maximum of eight specimens. The permeability test ran for at least 50 hr. and thus the frequency with which the eight batches could be made was also controlled.

APPARATUS

Permeability Apparatus—The apparatus for measuring the flow of water into the permeability specimens is shown in Fig. 1. The smallest divisions on the scale represent 0.001 gal., and, by use of the sliding target, readings may be interpolated to 0.0001 gal., although in this test such precision was adhered to only when reading the dummy, the readings for the test specimens being made to 0.0005 gal. The coil of high resistance wire was bedded in portland cement mortar which was mixed quite wet and cured for several days in the moist closet. The coil was used to heat the casting.

The air pressure system included two reservoirs, distribution pipe, flexible copper connecting tubes, a pressure gage and the necessary needle valves. The reservoirs were of relatively large

capacity when compared to the distributing pipe and so insured an almost constant pressure during the run.

The dummy consisted of a test unit similar to those used for the actual testing with the exception that the dummy specimen was cast on the outside of an iron plate and this bolted to a casting (c). The volume of water in the casting was the same as in the regular test units.

A wet and dry bulb thermometer and an electric fan for directing a current of air against the wick of the wet bulb were a part of the apparatus.

Heater for Absorption Specimens—The absorption cylinders were dried in a double-walled gas heater that was insulated on the top and sides with an asbestos covering. A solution of glycerine and water, mixed in such proportions that it boiled at about 220 to 240 deg. F., filled the space between the walls. The heater was large enough to dry 48 cylinders at one time.

TESTING

Length of Curing Period—Specimens were ordinarily removed from the moist closet at an age of 28 days. The permeability specimens of the first few lots were removed three or four days earlier with the expectation that they would be ready for testing on the 28th day but this was found to be impossible, and for the balance of the test specimens of all three types were removed at an age of 28 days, except for six lots which were removed at 27 days, 11 lots at 29 days and one lot at 30 days.

Compression Cylinders—Compression cylinders were broken on a 200,000-lb. testing machine the day they were removed from the moist closet. The top of the cylinders as molded was capped with neat gypsum plaster and the bottom also similarly capped in the few cases where an examination showed that this surface was not plane.

Absorption Cylinders—When the absorption cylinders were removed from the moist closet, they were first cleaned with a fiber brush, and then weighed to 0.01 lb. and left to accumulate until there was room in the heater. After they had been dried to constant weight they were immersed in water at room temperature and weighed after ½, 6, 24 and 48 hr. They were then capped on both ends and broken the same day. The weight after 48 hours in water was approximately the same as when first re-

moved from the moist closet so that the compression and absorption cylinders contained about the same amount of water when broken.

Permeability Specimens—With the exceptions noted in the first paragraph of this section permeability specimens were removed from the moist closet at an age of 28 days. They were immediately brushed to remove any loose material and weighed in air and in water to secure data from which the density of the concrete could be calculated. One coat of an asphalt priming paint was applied to the cylindrical surface to aid in securing a watertight bond. It was found that the surface of the specimen was dry enough for the priming coat after remaining in the dry air of the laboratory for about three days and that the priming coat required about 24 hr. to harden.

The first operation in the sealing process was to place the casting in a horizontal position with the circular shelf that supported the outer face of the specimen on the bottom. A small amount of neat gypsum plaster was then spread on this shelf and the specimen immediately placed and centered in the casting. The plaster bed prevented the molten asphalt from escaping during the pouring and supported the specimen rigidly in the casting when the air pressure was applied. Specimens were so placed in the casting that the flow of water was from the bottom to the top as originally molded.

After the plaster bed had dried for at least a half hour the casting was heated by passing an electric current through the resistence coil for three to five minutes. This was done for two reasons; first, to prevent the molten asphalt from being chilled before it could completely fill the space around the specimen and, second, to improve the seal by compressing the asphalt as the casting shrank upon cooling. Crystal Steep Roofing asphalt was used.

After the asphalt was poured, the casting with the specimen sealed within it was bolted to the other part of apparatus, the soft rubber gasket having first been put in place. The permeability tube was then filled with water and the copper tubing carrying the air pressure to the top of each water column attached. After the air had been applied the valves between the air reservoirs and the rest of the apparatus were closed to determine

whether there were air leaks in the tubing or connections. The actual testing was begun after any such leaks had been stopped and the air valves opened. Readings were then taken of the height of the water in the permeability tubes, of the height of the water in the dummy unit, of the air pressure and of the temperatures as read from a wet and dry bulb thermometer. All of the readings were taken every time the height of the water in the permeability tubes was read.

Ordinarily readings were taken and the face of each specimen examined at intervals of one hour or so for the first few hours and afterwards at greater intervals. In general at least three readings were taken each day during the 50-hr. test period.

Changes in air pressure and in room temperature produced changes in the height of the water column. In order to correct for such changes the dummy readings were employed. It was found, however, that the corrections for such changes were necessary only in the less permeable specimens.

The room temperature and relative humidity were not considered in calculating the data, for they appeared to have no material effect on the flow of water through the specimen, although they probably had some effect on the appearance of the face where there was enough water flowing through to make the surface appear slightly damp.

The pressure on the top of the water column was maintained as nearly as possible at 40 lb. per sq. in. except in some of the later tests where 100 lb. was used. In most cases it varied only slightly during the run. The increase in pressure on the specimen due to the height of water in the tube was not considered, for the differences in water flow due to such small changes in pressure are much less than the difference regularly found between two specimens of the same mix.

Untreated city water was used, it being considered that the test period was too short to permit clogging of the voids in the concrete by sediment in the water or by bacterial action. It should be noted that the apparatus was completely drained of water between runs and a fresh supply used with each set of specimens.

While no attempt was made to measure the water flowing from the concrete, the face of each specimen was carefully examined at

each reading and its condition noted on the data card. A defective seal could be detected by the appearance of water at the edge of the specimen. In such cases a notation was made on the card and the specimen rerun at the first opportunity. Some of the specimens were plainly defective and such specimens were not considered in calculating the data.

At the end of the test period the air valve leading to the reservoirs was closed and the casting containing the specimen removed from the apparatus. The casting was then turned so that the specimen would drop out when the asphalt was melted by passing an electric current through the heating coil imbedded in the casting. The inside of the casting was scraped to remove the asphalt and the gypsum that did not come out with the specimen and it was then ready for the next specimen.

COMPUTATIONS

Permeability—The data on the flow of water was plotted on graph paper, the flow being plotted against time, and the resultant curve corrected when necessary from readings of the dummy. The 20, 40 and 50-hr. flow was read from the curve and listed in a table. The flow for the 20 to 50 and 40 to 50-hr. intervals was thus obtained, the units being gal. per sq. ft. per hr. The values are given in Table 8. Whenever the test results for any one specimen did not conform more or less to the average of other specimens of the same mix, the data were discarded and the specimen rerun at a later time. In a few instances so much difficulty was experienced in obtaining data that only three values were used in the average, but this was true in only about 5 per cent of the cases. About 60 per cent of the results are the averages for five values and 35 per cent the average of four values.

Density—In computing the density of the individual batches the weights of the permeability specimen in air and in water were used together with the total weight of the batch, including the water, and the specific gravity of the cement, sand and gravel. The following formula was used:

$$\rho = \frac{\left[\frac{W_c}{62.5K_c} + \frac{W_s}{62.5K_s} + \frac{W_g}{62.5K_g}\right]\frac{W_p}{W_b}}{\frac{W_p - W_{pw}}{62.5}}$$

ρ —Density or absolute volume of cement, sand and gravel per unit of concrete volume.
W_c —Weight of cement used in batch.
W_s —Weight of sand used in batch.
W_g —Weight of gravel used in batch.
W_p —Weight of permeability specimen in air.
W_{pw}—Weight of permeability specimen in water.
W_b —Weight of batch including water.
K_c —Specific gravity of cement.
K_s —Specific gravity of sand.
K_g —Specific gravity of gravel.

It will be noted that the first term in the brackets gives the absolute volume of cement in the batch. The absolute volume of cement per cubic foot of concrete was obtained by dividing the absolute volume of cement in the batch by the volume of the batch. This latter value was computed from the weight of the permeability specimen and of the batch and the volume of the permeability specimen.

Absorption—The absorption was calculated for the ½, 6, 24, and 48-hr. intervals using the oven dry weight of the specimen as a basis. The results for the five specimens of any one mix and consistency were averaged and these results are listed in Table 8. In a few cases, when readings were not made exactly on schedule, the results of the individual specimens were plotted and an average curve drawn through these points. The data were then obtained from this graph.

Water-Cement Ratio—The water-cement ratio was computed for the first series and in the four succeeding ones it was kept constant, the amount of mixing water required for the batches being calculated in advance after the moisture content of the aggregate had been determined. A correction was made for the absorption of the aggregate in all of the computations. At the beginning of the second series the total absorption for the aggregate was not known exactly and so the same amount of water was used that was used in the first series. Later, when the water-cement ratios for these batches had been computed, it was found that they were almost exactly the same as those of the first series. The average values are shown in the tables.

RESULTS OF TESTS

Significance of Data from these Permeability Tests—When considering tests of this sort it should be remembered that laboratory tests on the permeability of concrete are qualitative rather than quantitative and that differences between types of apparatus, size and shape of specimens and other differences in the manner of performing the tests make it impossible to compare directly the rate of water flow through concrete of a given composition as determined by different laboratories. Also, it hardly seems possible to reproduce conditions in an actual structure with the small specimens of the laboratory but in a properly conducted laboratory it should be possible to compare the permeability of different concretes under laboratory conditions. It would seem reasonable that of two different concretes the one that showed the greater leakage in the laboratory would also show the greater leakage in an actual structure, but it should not be expected that a concrete which showed water on an exposed face in the laboratory would necessarily do so in an actual structure, or vice versa. In these tests the relative permeabilities of different concretes are compared under laboratory conditions, these conditions having been maintained as nearly constant throughout the test as was possible.

In these tests the flow of water reported represents flow into the specimen and thus includes both the water absorbed by the specimen and that flowing through the specimen. The flow curves plotted from the data cards showed a high initial rate of flow whch gradually decreased as the run continued and the specimen approached the saturation point. The absorption data reported on cylinders from the same concrete showed there was little or no increase in weight after the cylinders had been immersed in water for even 24 hr. so that it is reasonable to suppose that very little of the flow noted for the 40 to 50-hr. period represented water absorbed by the specimen. In Table 10 it can be seen that there is some difference in the flow for the 40 to 50 and 100 to 110-hr. intervals, but since this represents flow under 100 lb. per sq. in. pressure the differences are probably greater than they would be under lower pressure. None of the concrete was wholly impermeable as far as flow into it was concerned, even though the exposed face apparently was perfectly dry. In such cases the

rate of flow for the 40 to 50-hr. period was very low, being less than 0.001 gal. per sq. ft. per hr. in all cases and much less than that in most cases where there was no sign of moisture during the test. Such small amounts of water might readily evaporate on reaching the free surface. As might be expected the flow curves showed that the rate of flow was decreasing slightly even at the end of the 50-hr. period. It has been shown in other tests that the better the curing and the longer the curing continues the less permeable the concrete becomes. The curing conditions during the run were most excellent so that a continuous but very slight decrease in flow might be expected.

Table 8 gives the rate of flow for both the 20 to 50-hr. period and the 40 to 50-hr. period, but only the flow for the latter period will be considered for this report.

For various reasons it was impossible to test all specimens at the same age, but they were removed from the moist room to the dry air of the laboratory at an age of 28 days (a day earlier or later in a few cases as noted in a previous section). These tests and those reported by others indicate but little decrease in permeability after proper curing is discontinued so that the slight average differences in age at the time of testing were disregarded.

It would seem that the method used in these tests of measuring the water flowing into the concrete is superior to those methods which measure the water flowing from the concrete. Concrete of ordinary mixes and properly cured for 28 days is so impermeable that no measurable amount of water escapes from the exposed face, making it necessary to use either much leaner mixes than are ordinarily used in the field or to test the specimens at a very early age. Furthermore there is no way of differentiating between concretes which appear dry on the exposed faces or between concretes permeable enough for the exposed faces to be wet but with not enough flow for it to be measurable. With the method used in these tests any concrete may be tested at any age and the flow of water into the specimen measured.

Relation of Flow to Water-Cement Ratio—Figs. 2 and 3 show flow plotted against water-cement ratio. In Fig. 2, this relation for the 10 most impermeable mixes, is so shown that it is possible to tell at a glance whether the exposed face was dry, somewhat wet or quite wet. In some cases the mix with the lowest water-

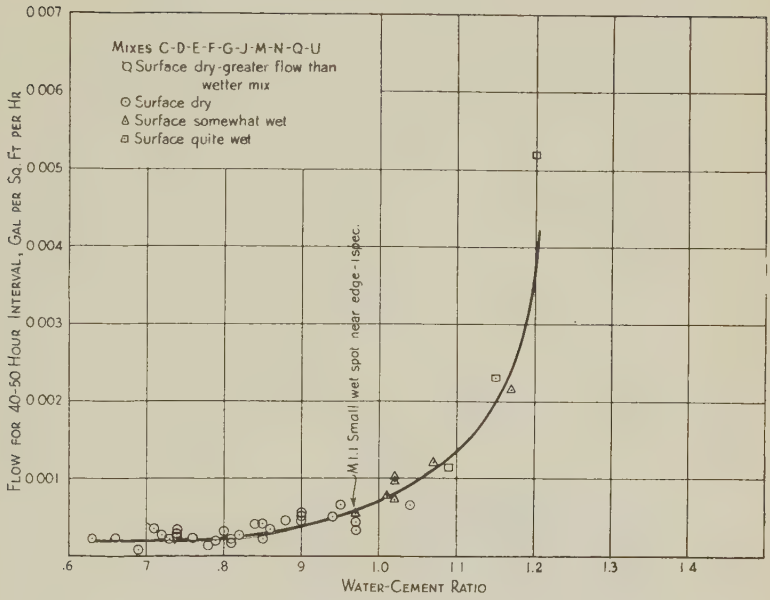


Fig. 2—Relation of flow to water-cement ratio for most impermeable mixes

cement ratio had a greater flow than the same mix with a higher water-cement ratio, although in neither case was there any indication of water on the exposed face of the specimen. Fig. 3, showing the relationship for all mixes, indicates that the correlation between water-cement ratio and flow is only general, and that such other properties as grading of aggregate, richness of mix and consistency affect this relationship to a considerable degree. For instance, of the 10 most impermeable mixes shown in Fig. 2, mixes E, F, G, M and N with 40 per cent Janesville sand and a fineness modulus of 5.5 had about the best grading of aggregate, while mixes C, D, J, Q and U were rich mixes of poorer grading, indicating that with the best grading of aggregate an impermeable concrete can be obtained with a rather lean mix, but with a poorer grading of aggregate a rather rich mix is required.

From a study of Fig. 3 and Table 8 it can be seen that for these tests the exposed face of the specimen was at least somewhat wet whenever the water-cement ratio was over 1.0 (7.5 gal. per sack of cement) with the single exception of mix F-1.2, and that with

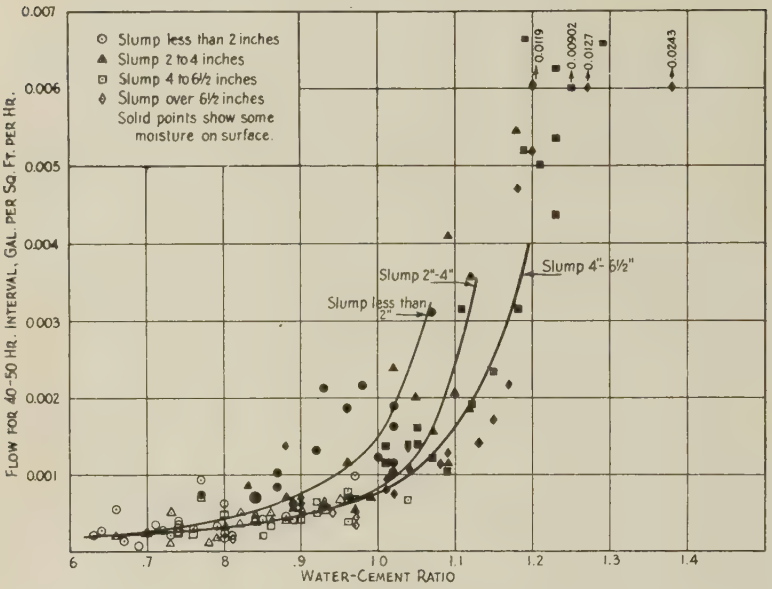


FIG. 3—EFFECT OF WATER-CEMENT RATIO AND SLUMP ON FLOW FOR ALL MIXES

such poorly graded mixes as Y-1.0 and a-1.0 the exposed face was wet although the water-cement ratio was much lower. In Fig. 3 the solid points indicate that the exposed surface was somewhat moist or wet, and their position shows that when the flow was less than 0.0005 gal. per sq. ft. per hr. the exposed face was dry for all mixes, but when the flow was 0.001 gal. per sq. ft. per hr. or more, the exposed face was at least somewhat wet for all mixes.

It might be mentioned at this point that mix R, which was the strongest concrete tested, showed up very badly in all cases where flow was plotted against water-cement ratio, strength or other variables. There was unusual variation between the flow of the different specimens of this mix but not enough to justify discarding any of them from the calculations. It would seem that the mix R (1:4 by volume) should have given at least as good results as mix Q (1:5 by volume) which had the same grading of aggregate. The densities for the two mixes were almost the same.

Relation of Flow to Consistency—This relationship is shown to a certain extent by Fig. 3 but more accurately by Fig. 4. Fig. 3

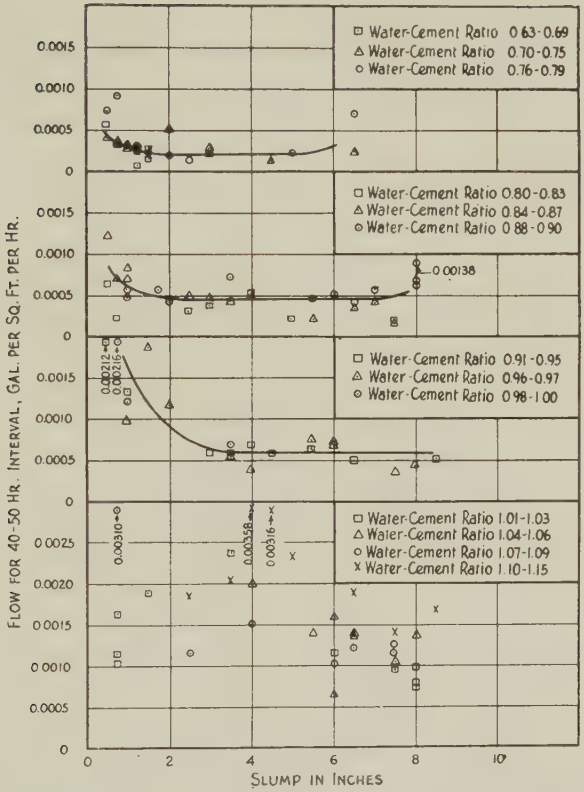


FIG. 4—EFFECT OF CONSISTENCY ON FLOW FOR MIXES OF APPROXIMATELY THE SAME WATER-CEMENT RATIO

indicates that the less permeable mixes are those with low water-cement ratios and that it is possible to have rather impermeable mixes with relatively high slump. The scattering of points made it difficult to draw accurate curves or definite conclusions from Fig. 3. Fig. 4 shows more clearly that the most impermeable mixes are obtained with low water-cement ratios when the slump is at least 2 in., that for higher values of water-cement ratios less impermeable mixes are obtained, and that to obtain the most impermeable mixes for a given water-cement ratio the minimum permissible slump must be increased with increase of water-cement ratio. For very high water-cement ratios the scattering

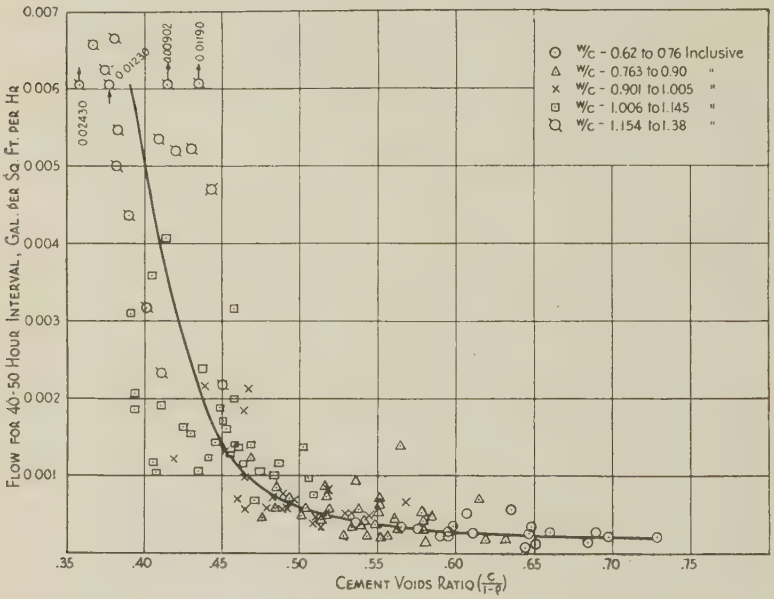


FIG. 5—EFFECT OF CEMENT-VOIDS RATIO AND WATER-CEMENT RATIO ON FLOW FOR ALL MIXES

of points indicates that definite control over the properties of the mix is lost and that, in general, high slumps are required for the most impermeable concrete. It should be remembered that all mixes used in these tests had sufficient water to make them plastic and workable and that those mixes with a slump of less than 2 in. obeyed the water-cement-strength law even though the flow of water through them was, in most cases, greater than mixes of slightly higher slump. In 15 cases the 1.1 mix showed a lesser flow than the 1.0 mix and in 9 of these mixes the average slump for the 1.1 mix was from 2 to 3 in. In 19 cases the 1.1 mix showed a greater flow than the 1.0 mix and in 12 of these the average slump was over 3 in. The water-cement ratio of the 1.1 mix was 10 per cent greater than that of the 1.0 mix, while the flow of water was less for the 1.1 mix in the majority of cases where the slump of the 1.1 mix was from 2 to 3 in. This confirms what has been found in other permeability tests; that a wetter mix is required for minimum flow than is required for maximum strength.

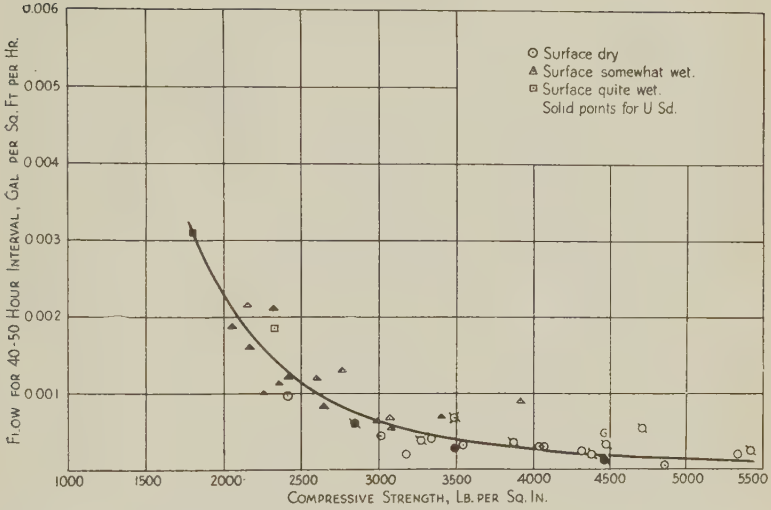


FIG. 6—RELATION OF COMPRESSIVE STRENGTH TO FLOW FOR MIXES WITH SLUMP LESS THAN 2 INCHES

Relation of Flow to Cement-Voids Ratio—Fig. 5 shows flow plotted against cement-voids ratio for all mixes, and with varying ranges of water-cement ratio. From this figure it would seem that there is a slightly more definite relationship between flow and cement-voids ratio than between flow and water-cement ratio. It is reasonable that this should be the case, for the grading of aggregate and consistency, which are important in influencing the flow affect the value of the cement-voids ratio but not the water-cement ratio. The slump, however, had some effect on the relation of flow to cement-voids ratio, for when the data were separated into four slump ranges and plotted, the curve for the 2 to 4 in. slump was farthest to the left.

Relation of Flow to Compressive Strength—Figs. 6 and 7 show flow plotted against compressive strength for two ranges in slump; the former figure for mixes with slump less than 2 in. and the latter for mixes with slump over 6½ in. Space does not permit publication of the curves for mixes with slumps from 2 to 4 in. and from 4 to 6½ in. These two curves lie within those shown in Figs. 6 and 7, with a shift to the left as the slump increases. There is apparently a somewhat closer relationship

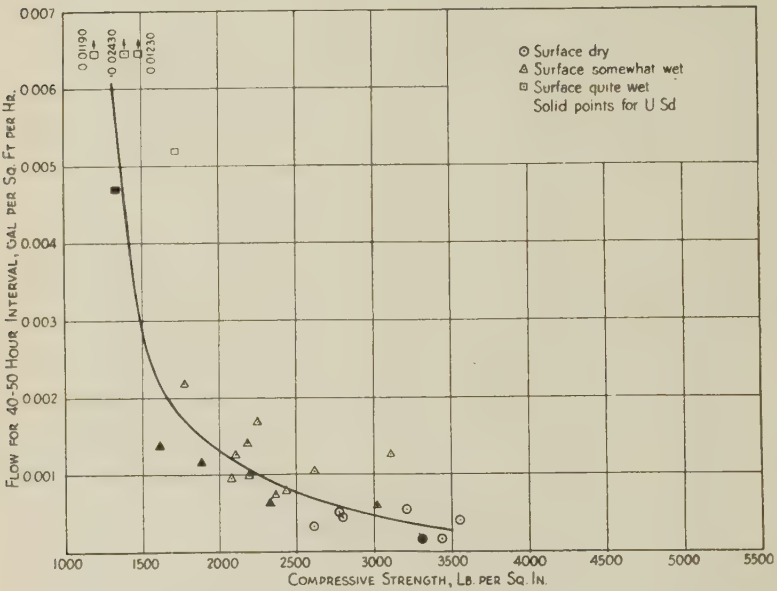


FIG. 7—RELATION OF COMPRESSIVE STRENGTH TO FLOW FOR MIXES WITH SLUMP OVER 6½ INCHES

between flow and strength than between flow and water-cement ratio. In these tests only one mix having a strength of over 3500 lb. per sq. in. showed any moisture on the exposed face, while all mixes with a strength of less than 2250 lb. per sq. in. showed some moisture on the exposed surface.

Relation of Flow to Proportion of Cement—Fig. 8 shows the cement-dry aggregate ratio (by weight) plotted against flow for mixes with a 2 to 4-in. slump. Average curves for mixes with other ranges of slump lie to the right of the one shown in the figure. They indicate that, in general, rich mixes are most impermeable.

Relation of Flow to Grading Aggregate—Fig. 9 shows flow plotted against fineness modulus for various proportions of cement and for two ranges of slump. The 1-4 and 1-5 mixes by volume agree with what has been shown already; i. e., that the more impermeable mixes are those with slumps greater than that required for normal consistency. The 1-6 mixes, however, show a marked

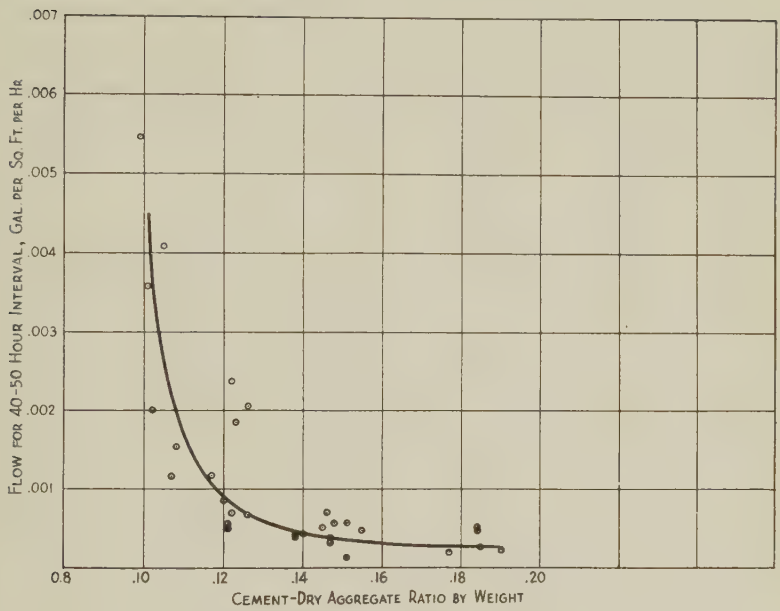


FIG. 8—RELATION OF CEMENT-DRY AGGREGATE RATIO TO FLOW FOR MIXES WITH 2 TO 4-INCH SLUMP

difference. In plotting this graph it was very difficult to obtain good curves unless the slump was restricted to narrow limits, and naturally only a few mixes could be used. This curve is not entirely irreconcilable with what is shown in Fig. 4, for it represents lean mixes of poor or fair grading, and it might be expected that for these the most impermeable concrete is that with only enough water to make it plastic. Fig. 9 indicates that as mixes become leaner, the fineness modulus must increase if minimum flow is desired. Roughly this variation is from a fineness modulus of 5.2 for a 1-4 mix to 5.8 for a 1-6 mix by volume.

The previous paragraphs indicate that a well graded aggregate is even more important from the standpoint of water tightness than it is from the standpoint of strength. A study of the grading of aggregate for the 10 most impermeable mixes shown in Fig. 2 indicate that when large sizes of coarse aggregate predominate, as for mixes C, D, E, F, G, J, M and N, rich mixes like 1-4 or 1-5 should be used when the mixes are slightly over or under sanded

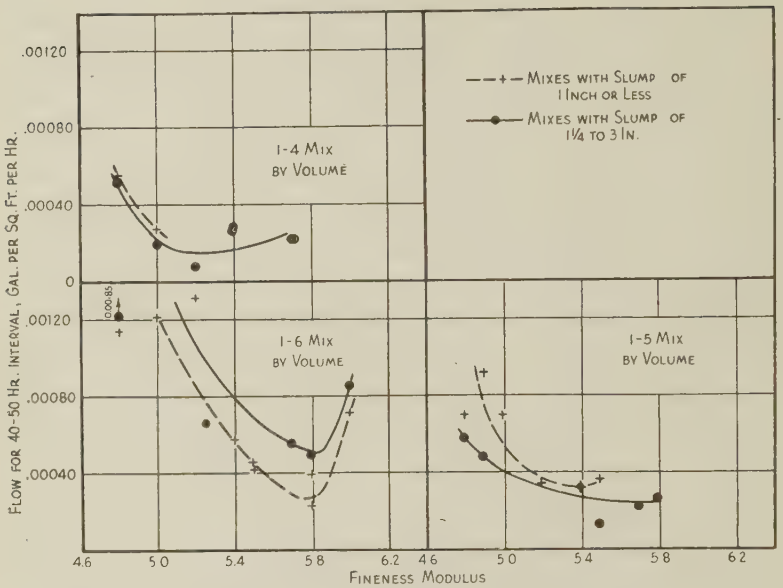


FIG. 9—RELATION OF FINENESS MODULUS TO FLOW FOR MIXES WITH SLUMP LESS THAN 3 INCHES

(50 or 35 per cent sand), but where the proportion of sand is 40 per cent by weight of the mixed aggregate the leaner mixes like 1-5 or 1-6 may be used. When the smaller sizes of coarse aggregate predominate, as in mixes Q and U, a 1-5 mix can be used with 35 per cent sand but this mix must be made richer when a greater proportion of sand is used.

Relation of Compressive Strength to Absorption—Fig. 10 shows this relationship for Janesville sand mixes. The one for University sand was similar but is not shown. In general the higher values of fineness modulus showed least absorption, but there was little difference for the best concretes.

Relation of Water-Cement Ratio to Absorption—Fig. 11 shows this relationship for mixes with a 1½ to 3-in. slump. In plotting this graph, as in some previous cases, it was necessary to restrict the consistency to narrow limits in order to obtain a satisfactory curve. As might be expected the absorption increases with an increase in water-cement ratio.

Relation of Flow to Absorption—Graphs showing this relation-

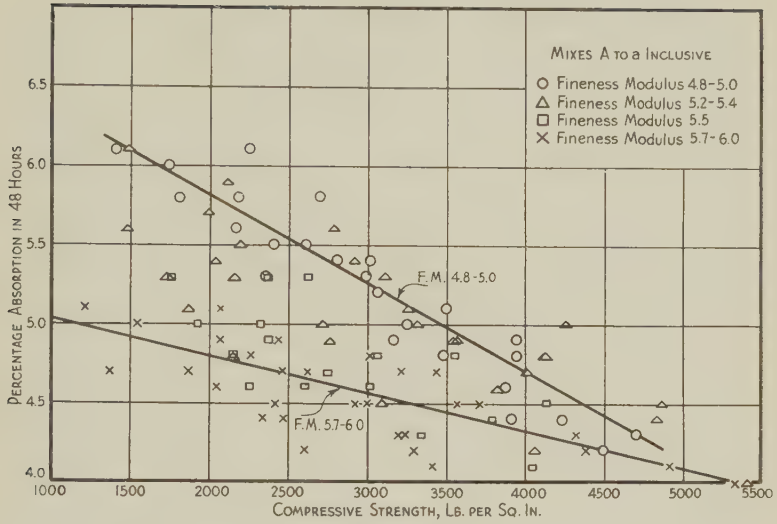


FIG. 10—RELATION OF COMPRESSIVE STRENGTH TO ABSORPTION FOR JANESVILLE SAND MIXES

ship were plotted but the individual points were spread too much to draw curves through them. Those mixes with the least flow had least absorption. This agrees with Fig. 12 which shows that mixes with greater density had least absorption.

Relation of Strength to Water-Cement Ratio—Fig. 13 shows the familiar relationship of strength to water-cement ratio. It is to be noted that the University sand mixes have less strength for given values of water-cement ratio than the Janesville sand mixes. The University sand was very fine and poorly graded, hence mixes containing it had lower strength than similar mixes of Janesville sand.

High Pressure Test Results –Some six months after the testing had been finished at the 40 lb. per sq. in. pressure 19 mixes were selected as representative ones and tested for 110 hr. at 100 lb. per sq. in. pressure. The results of these tests are shown in Table 10 and Fig. 14. The age of the specimens when first tested at the lower pressure averaged about 40 days and that of seven of these when retested about 150 days. The age when tested at the higher pressure averaged about 14 months. Fig. 14 shows, how-

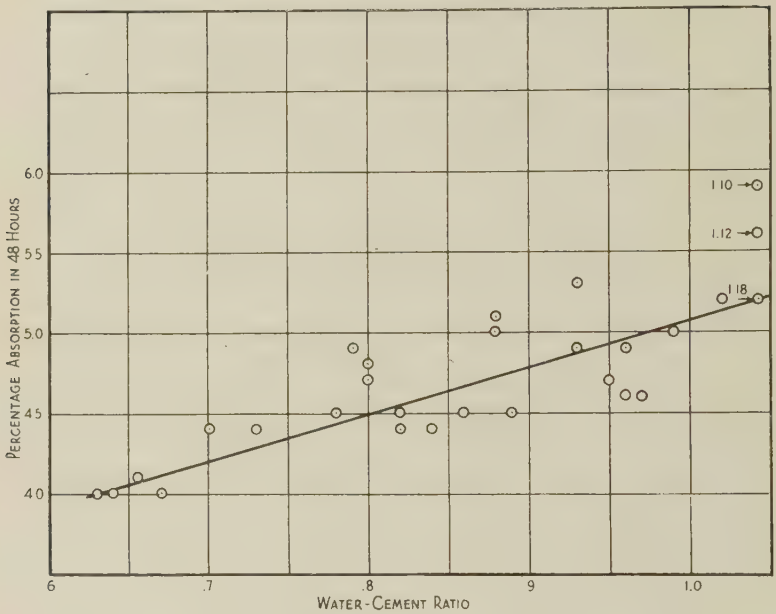


FIG. 11—RELATION OF WATER-CEMENT RATIO TO ABSORPTION FOR MIXES WITH 1½ TO 3-INCH SLUMP

ever, that the flow at the lower pressure of the 40 to 50-hr. interval is about 60 per cent of that for the higher pressure. Had this second series of tests been made at the lower pressure the two curves in the left half of the figure would more nearly coincide. It is evident, therefore, that an increase in age after 40 days has only slight effect on the watertightness. The right hand half of the figure shows the results at the higher pressure for the 100 to 110-hr. interval. It is interesting to note that this curve and the one for the low pressure in the other half of the figure are coincident up to a water-cement ratio of 0.90, and that after a water-cement ratio of 1.00 the curve for the higher pressure increases more sharply. One of the 19 mixes (X-1.35) that had shown moisture on the exposed surface in the original tests appeared dry at the higher pressure, while four of the mixes (F-1.1, G-1.1, U-1.2 and X-1.0) which had previously appeared dry now showed some moisture on the surface.

Long-Time Test Results—Fig. 15 shows flow plotted against

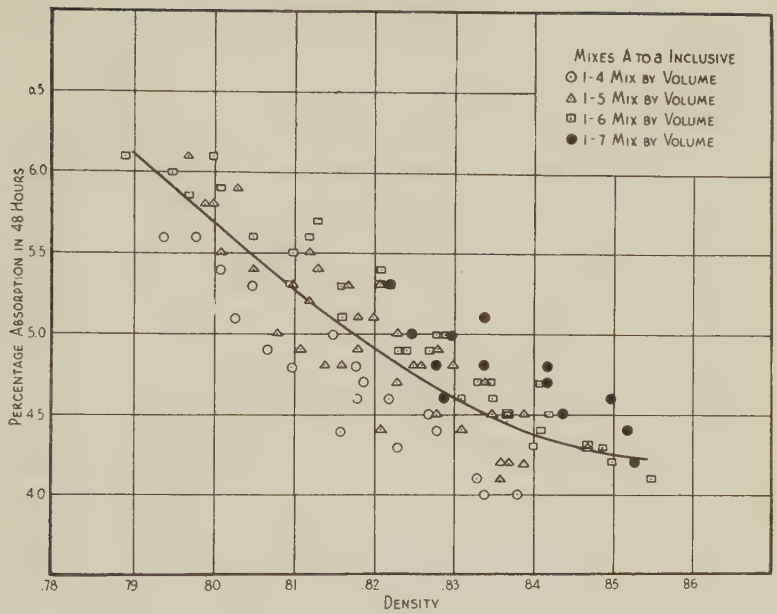


FIG. 12—RELATION OF DENSITY TO ABSORPTION FOR JANESVILLE SAND MIXES

time for several mixes. These tests were made in three installments; the first for 500 hr., the second for 1000 additional hours and the third for 3000 more hours. The dotted portions of the curves indicate these breaks in the testing. About five months elapsed between the first two runs but the specimens were not removed from the apparatus during this interval. The water remained in the casting; the air pressure was turned off. At the end of a total of 1500 testing hours various indigo dyes were added to the water in the hope of having the coloring show on the exposed surface but none was noticeable after 3000 hr., although the slopes of the curves remained constant. It may be possible that the voids in the concrete act similarly to those in charcoal and that the dyes were absorbed as are poisonous gases by the charcoal. Unfortunately the glass water tube on the casting holding the specimen 5-L1.0 was broken at the start of the run. Mix 4-X1.35 was not tested in the original 500-hr. run.

This long time testing was carried on to determine whether or not the flow of water into the specimen would continue only until

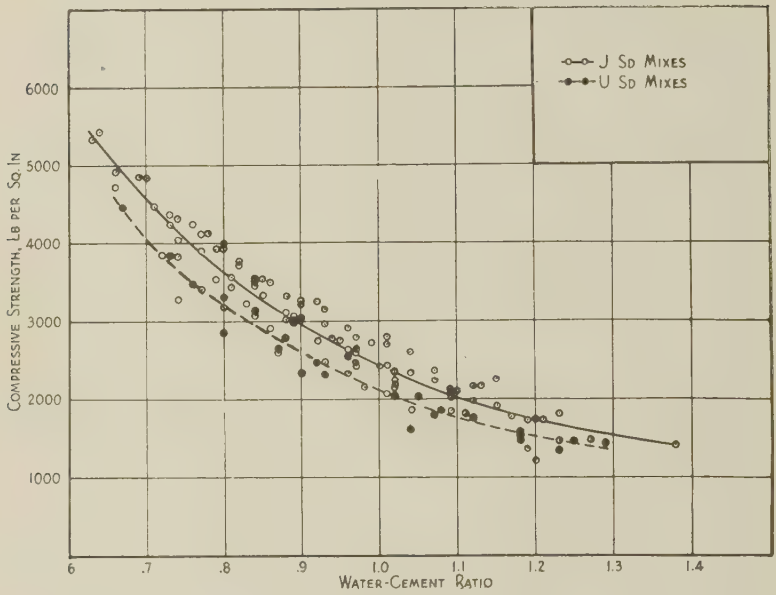


FIG. 13—RELATION OF WATER-CEMENT RATIO TO COMPRESSIVE STRENGTH FOR ALL MIXES

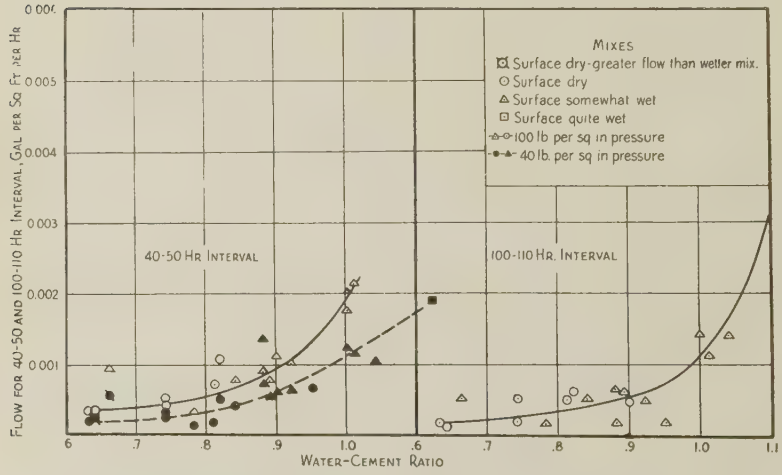


FIG. 14—EFFECT OF VARYING PRESSURE AND TIME ON THE WATER-CEMENT RATIO—FLOW RELATIONSHIP

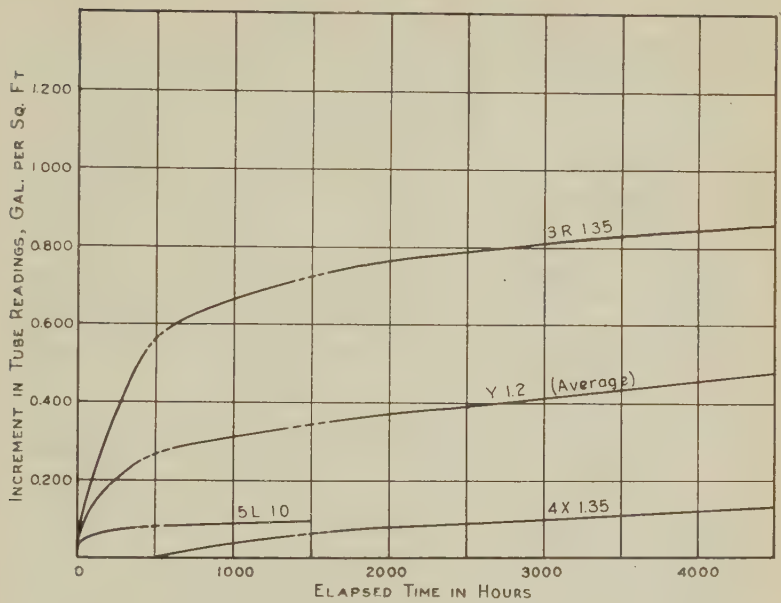


FIG. 15—TIME-FLOW CURVE FOR VARIOUS MIXES UNDER 100 LB. PER SQ. IN. PRESSURE

the voids had been completely filled, and whether the voids would become clogged by sediment or bacterial action in the water. It is evident from the figure that some clogging of the voids did take place but that it was not enough to prevent flow through the specimen at 100 lb. per sq. in. pressure.

The following calculations show the amount of water that must have passed through the specimen during the test run. Tests reported by F. R. McMillan in "Basic Principles of Concrete Making" (p. 9-11) indicated that approximately 0.20 of the volume of cement was combined water and this figure was used in the following calculations.

Mix 3-R-1.35

Permeability Specimen		Materials in Mix	
Weight	36.65 lb.	Cement	19.5 lb.
Volume	0.243 cu. ft.	Cement and Agg.	125.5 lb.
Density	0.805	Water used	12.64 lb.
		Absorption	1.35 lb.
			0.188 cu. ft.

$a + b + c = (0.805 \times 0.243) =$

$$\text{Vol. water combined } \frac{36.65}{138.1} \times \frac{19.5}{94} \times 0.20 = 0.011$$

Vol. water voids $\frac{36.65}{138.1} \times \frac{11.3}{62.5} - 0.011 = 0.037$

Abs. vol. cement, agg., combined water and water voids	0.236 cu. ft.
Vol. of water entering specimen (0.431 gal.)	0.0575
	0.2935
Vol. of specimen	0.243
Vol. of water passing through specimen in 4500 hours	0.0505 cu. ft.

Mix Y 1.2

Permeability Specimen		Materials in Mix	
Weight	36.05 lb.	Cement	19.5 lb.
Volume	0.236 cu. ft.	Cement and Agg.	126.5 lb.
Density	0.801	Water used	12.57 lb.
		Absorption	.89 lb.

Abs. vol. cement agg., combined water and water voids	0.239 cu. ft.
Vol. of water entering specimen (0.236 gal.)	0.0315
	0.2705
Vol. of specimen	0.236
Vol. of water passing through specimen in 4500 hrs.	0.0345 cu. ft.

Mix 5 L 1.0

Permeability Specimen		Materials in Mix	
Weight	38.10 lb.	Cement	15.7 lb.
Volume	0.247 cu. ft.	Cement and Agg.	124.5 lb.
Density	0.847	Water used	8.76 lb.
		Absorption	.89 lb.

Abs. vol. of cement, agg., combined water and water voids	0.244 cu. ft.
Vol. of water entering specimen	0.0066
	0.2506
Vol. of Specimen	0.247
Vol. of water passing through specimen in 1500 hrs.	0.0036 cu. ft.

Undoubtedly some water was lost in the drying of the specimens before they were tested in the later runs. During this interval they were stacked either out of doors or in the dry air of the laboratory with their cylindrical sides still covered with asphalt. It is probable that they lost about 1 per cent of moisture by weight. Tests on absorption indicate, however, that concrete will absorb approximately 1½ per cent by weight when immersed in water for a half hour (see Table 8). Since at least a half hour elapsed after the permeability tubes had been filled with water and the pressure turned on and before the readings were taken, the evaporation was counterbalanced by the absorption (under pressure) and no account of it was taken in the preceding calculations. These computations together with the constant slope of the curves of Fig. 15 indicate that water will flow through even

good concrete (5 L-1.0 mix) although the flow may be slow enough so that the surface appears dry.

SUMMARY

The results of these tests, have been analyzed in the previous section and may be summarized as follows:

1. It might be expected that the permeability would increase as the water-cement ratio increased and these tests indicate that there is such a relationship. However, this relationship is influenced considerably by the consistency and, especially with the leaner mixes, by the grading of the aggregate. More water is required for minimum permeability than for maximum strength, the permeability increasing when the water is reduced below the amount required for a slump of about 2 to 3 in.

2. Permeability decreases as strength increases and these tests indicate that this relationship may be somewhat closer than that between the water-cement ratio and permeability, although there is no apparent reason why this should be so if the relationship between water-cement ratio and strength is as fundamental as it is generally believed to be.

3. Permeability decreases as the cement-voids ratio increases and this relationship also seems to be more definite than that bebetween the water-cement ratio and permeability, apparently because of the effect that the grading of the aggregate and the consistency have on the cement-voids ratio.

4. These tests indicate that there is no relationship between permeability and absorption, except the very general one that a good concrete might be expected to be lower both in permeability and absorption than a poor concrete.

5. These tests indicate that for the comparatively small specimens used there is little change in permeability after the concrete is removed from the moist room into a dry atmosphere, and that even considerable differences in age at time of testing are immaterial, if the age at which the specimens are removed from the moist room is kept constant. In the average structure the concrete as a whole would not dry out so rapidly and the concrete might be expected to decrease in permeability for a considerable time. It may well be, however, that the concrete near the surface may be affected in the same way as the laboratory specimens, and

this would be of considerable importance if such a surface was exposed to the weather and to repeated freezings and thawings.

6. Tests at higher pressure indicate that more water will flow through the specimen, but the increased flow is not directly proportional to the added pressure. Long time tests under high pressure show that clogging of the void spaces is not enough to prevent flow through concrete although the flow may be slow enough so that the surface appears dry.

TABLE 1—CHEMICAL ANALYSIS OF CEMENT—MANUFACTURERS' TESTS

Kind of Cement	SiO_2	Fe_2O_3	Al_2O_3	CaO	Mg O	SO_3	Mn O	Loss on Ignition	Total
Atlas	21.72	2.48	5.68	62.98	3.17	1.89	....	0.99	98.91
Lehigh	21.82	2.45	5.87	62.76	2.69	1.73	....	....	97.32
Universal	22.00	2.05	5.59	64.50	1.96	1.62	0.82	1.53	100.04

TABLE 2—PHYSICAL TESTS OF CEMENT

Property	Mftrs. Reports			University of Wis. Tests			
	Atlas	Lehigh	Univ.	Atlas	Lehigh	Univ.	Mixture
Normal Consistency	22.0	22.0	....	23.0	23.0	23.0	23.0
Initial Set*	2:35	4:15	3:05	3:05	4:10	4:00	....
Final Set*	5:15	6:50	....	7:45	7:35	8:35	....
Soundness	OK	OK	OK	OK	OK	OK	OK
Fineness	92.8	82.5	84.2	93.0	82.7	85.9	87.7
Tensile Strength 1-3 Std. Mortar Briquets							
1 day		95					
3 day		192		287	236†	239	305†
7 day	355	283	290	394	315	325	323
28 day		383	385	493	416	428	456
60 day				479	420	471	480
180 day				449	387	470	448
360 day				433	320	302	372
Specific Gravity							3.13
Compressive Strength 1-3 Std. Mortar 2 in. x 4 in. Cylinders							
7 day	3408						
14 day							3090
30 day							3228
62 day							3960
180 day							4114
360 day							4350
Tensile Strength Neat Cement Briquets							
1 day				453	297	285	468
7 day				786	767	736	700
28 day							776
60 day							701
360 day							601

*Gillmore Needle.
†4-day strength.

TABLE 3—PHYSICAL PROPERTIES OF SANDS

Kind	Fineness Modulus	Wt. per Cu. Ft.	Per Cent Voids	Absorption by Wt. of Sand Per Cent	Sp. Gr.
Janesville Sand	2.73	115	32.7	0.78	2.70
University Sand	1.53	108	34.0	0.40	2.64

TABLE 4—SIEVE ANALYSIS OF SANDS

Kind	Per Cent Coarser by Weight Number of Sieve					
	4	8	14	28	48	100
Janesville Sand	1.5	14.6	32.5	47.6	80.0	97.0
University Sand	0	0	2.0	8.0	52.0	91.0

TABLE 5—PHYSICAL PROPERTIES AND GRADING OF COARSE AGGREGATES

Gravel for Mixes	Parts by Weight—Size				Physical Properties			
	¼–⅜	⅜–¾	¾–1½	1½–2	F. M.	Absorption by Wt. of Gravel Per Cent	Sp. Gr.	Sp. Wt. lb. per cu. ft.
A to J, b to k	1	2	2.5	–	7.3	1.59	2.68	107.7
K to N	1	2	4	–	7.4	1.55	2.68	106.5
O to X	2	2	1	–	6.8	1.71	2.67	108.6
Y	1	2	–	–	6.7	1.75	2.67	105
Z to a	1	2	2	2	7.7	1.55	2.68	110

TABLE 6—GRADING OF MIXES

Mix	Mix by Volume		Per Cent Fine Agg. by Wt.	Per Cent Coarse Agg. by Wt.	Mix by Wt.	Wt. per Cu. Ft. lb.	F. M.
A	1:7	1:2.65:5.26	35	65	1:3.24:6.01	125	5.7
B	1:6	1:2.27:4.52			1:2.78:5.18		
C	1:5	1:1.89:3.76			1:2.32:4.31		
D	1:4	1:1.51:3.00			1:1.85:3.44		
E	1:7.09	1:3.13:4.86	40.7	59.3	1:3.84:5.55	125	5.5
F	1:6	1:2.60:4.16	40	60	1:3.18:4.77		
G	1:5	1:2.16:3.47			1:2.65:3.98		
H	1:5.62	1:3.25:3.52	50	50	1:3.99:3.99	133	5.0
I	1:4.69	1:2.71:2.93			1:3.32:3.32		
J	1:4	1:2.31:2.50			1:2.84:2.84		
K	1:6	1:2.37:4.76	35	65	1:2.90:5.38	130	5.8
L	1:5	1:1.98:3.97			1:2.42:4.50		
M	1:6	1:2.71:4.40	40	60	1:1.31:4.97	130	5.5
N	1:5	1:2.26:3.66			1:2.76:4.15		
O	1:7	1:2.73:5.37	35	65	1:3.34:6.20	128	5.4
P	1:6	1:2.34:4.60			1:2.88:5.35		
Q	1:5	1:1.95:3.83			1:2.39:4.43		
R	1:4	1:1.56:3.06			1:1.91:3.53		
S	1:6	1:2.69:4.28	40	60	1:3.29:4.94	129	5.2
T	1:5	1:2.24:3.57			1:2.74:4.12		
U	1:4	1:1.79:2.86			1:2.19:3.32		
V	1:6	1:3.33:3.52	50	50	1:4.08:4.08	128	4.8
W	1:5	1:2.77:2.94			1:3.39:3.39		
X	1:4	1:2.22:2.35			1:2.72:2.72		
Y	1:5	1:2.39:3.20	45	55	1:2.92:3.56	122	4.9
Z	1:7	1:2.80:5.44	35	65	1:3.44:6.40	132	6.0
a	1:6	1:2.40:4.66			1:2.93:5.46		
b	1:7	1:2.60:6.08	30	70	1:2.98:6.98	134	5.6
d	1:6	1:2.23:5.22			1:2.56:6.00		
e	1:5	1:1.86:4.35			1:2.14:5.01		
f	1:4	1:1.49:3.48			1:1.71:4.00		
g	1:7	1:3.08:5.73	35	65	1:3.52:6.55	136	5.3
h	1:6	1:2.65:4.92			1:3.01:5.58		
k	1:5	1:2.21:4.10			1:2.53:4.69		

TABLE 7—RESULTS OF COMPRESSION TESTS

Mark	Mix by Volume	Percentage of Sand	Water-Cement Ratio	Slump in in.	Flow Factor	Compressive Strength in lb. per sq. in.		Percentage Loss in Strength for Absorption Specimen	Age in Days of Absorption Specimen when Broken
						Normal at 28 Days	For Absorption Specimen		
A1.0	1:7	35	0.97	1	145	2410	2230	7.4	69
A1.1			1.07	4	170	2260	1730	23.4	69
A1.2			1.18	6½	200	1530	1410	7.8	62
B1.0	1:6	35	0.89	1¾	155	2990	2820	5.6	62
B1.1			0.96	5½	175	2620	2300	12.2	61
B1.2			1.05	6	195	2060	1810	12.2	61
B1.35			1.20	7½	225	1210	1070	11.5	64
C1.0	1:5	35	0.73	1¼	130	4380	4000	8.6	59
C1.1			0.81	5	185	3560	2940	17.4	60
C1.2			0.90	7	200	3210	2670	16.8	60
C1.35			1.01	8	230	2430	2000	17.7	59
D1.0	1:4	35	0.63	1½	125	5330	4980	6.5	59
D1.1			0.66	3	145	4920	4510	8.3	59
D1.2			0.74	6½	190	3830	3570	6.7	60
D1.35			0.81	7½	230	3430	3070	10.4	59
E1.0	1:7.1	40.7	1.02	¾	130	2250	2110	6.2	59
E1.1			1.09	2½	160	2140	1950	8.8	59
E1.2			1.15	5	180	1920	1630	15.1	57
F1.0	1:6	40	0.85	1	130	3340	3010	9.8	56
F1.1			0.95	4	170	2750	2450	10.8	56
F1.2			1.04	6	200	2320	2050	11.7	55
F1.35			1.17	7	230	1770	1390	21.5	55
G1.0	1:5	40	0.71	¾	115	4480	3690	19.8	53
G1.1			0.78	2½	145	4130	3450	16.6	53
G1.2			0.85	5½	185	3550	3000	15.4	56
G1.35			0.97	7½	225	2620	2150	17.8	55
H1.0	1:5.6	50	1.00	1	130	2410	2070	14.1	54
H1.1			1.10	3½	165	2100	1710	18.6	54
H1.2			1.21	6½	205	1740	1410	19.0	53
I1.0	1:4.7	50	0.84	1	125	3070	2610	15.0	53
I1.1			0.93	3½	165	2980	2520	15.4	53
I1.2			1.01	6½	225	2700	2130	21.1	53
I1.35			1.15	8½	245	2250	1800	20.0	53
J1.0	1:4	50	0.72	1	120	3870	3770	2.6	70
J1.1			0.79	2	145	3940	3590	8.8	74
J1.2			0.86	6½	195	3500	3210	8.2	74
J1.35			0.97	8	235	2790	2600	6.8	74
K1.0	1:6	35	0.74	½	120	3280	3170	3.3	59
K1.08			0.80	¾	130	3180	2840	10.7	61
K1.16			0.86	2½	155	2910	2540	12.7	60
K1.3			0.97	6	185	2460	2030	17.4	61
L1.0	1:5	35	0.74	1¼	130	4320	3740	13.3	55
L1.1			0.82	4	170	3710	3080	16.9	55
L1.2			0.89	6½	205	3010	2480	17.6	55
L1.35			1.01	7½	235	2070	1740	15.9	55
M1.0	1:6	40	0.88	1	120	3010	2740	8.9	54
M1.1			0.97	3½	155	2600	2510	3.4	56
M1.2			1.07	6½	210	2370	1960	17.3	56
M1.35			1.20	7	230	1730	1360	21.3	55

TABLE 7—RESULTS OF COMPRESSION TESTS—CONTINUED

Mark	Mix by Volume	Percentage of Sand	Water-Cement Ratio	Slump in in.	Flow Factor	Compressive Strength in lb. per sq. in.		Percentage Loss in Strength for Absorption Specimen	Age in Days of Absorption Specimen when Broken
						Normal at 28 Days	For Absorption Specimen		
N1.0	1:5	40	0.74	¾	120	4040	3670	9.2	58
N1.1			0.82	3	145	3780	3290	12.9	58
N1.2			0.90	6	195	3060	2790	8.8	73
N1.35			1.02	8	235	2360	2120	10.1	73
O1.0	1:7	35	0.98	¾	135	2150	1940	9.7	63
O1.1			1.09	2½	165	1870	1610	13.8	63
O1.2			1.19	5	185	1730	1370	20.8	63
P1.0	1:6	35	0.89	1	130	3080	2730	11.3	64
P1.1			0.99	3½	165	2710	2250	17.0	54
P1.2			1.09	6	195	2030	1740	14.2	54
P1.35			1.23	6½	215	1480	1230	16.9	54
Q1.0	1:5	35	0.74	1	125	4060	3780	6.7	54
Q1.1			0.80	2½	145	4000	3320	17.0	54
Q1.2			0.90	5½	190	3260	2760	15.3	54
Q1.35			1.02	8	225	2190	1800	17.8	54
R1.0	1:4	35	0.64	1½	125	5420	4940	8.7	52
R1.1			0.70	3	160	4830	4290	11.1	52
R1.2			0.77	6½	200	4120	3740	9.2	52
R1.35			0.88	8	240	3110	2580	17.0	52
S1.0	1:6	40	0.92	1	140	2760	2330	15.6	52
S1.1			1.02	3½	170	2160	1830	15.3	52
S1.2			1.12	6½	205	1990	1610	19.1	50
S1.35			1.27	8	240	1490	1190	20.1	50
T1.0	1:5	40	0.79	¾	130	3540	3070	13.2	52
T1.1			0.88	3½	165	3310	2980	9.9	58
T1.2			0.96	6	205	2920	2400	17.8	58
T1.35			1.09	7½	240	2110	1850	12.2	58
U1.0	1:4	40	0.69	1¼	135	4860	4420	9.0	58
U1.1			0.76	5	185	4250	3670	13.7	57
U1.2			0.84	7	215	3550	3290	7.3	57
U1.35			0.94	8½	245	2780	2460	11.5	57
V1.0	1:6	50	1.02	¾	130	2360	1960	17.0	59
V1.1			1.12	2½	160	2170	1840	15.2	53
V1.2			1.23	6½	200	1810	1500	17.1	53
V1.35			1.38	7½	240	1410	1230	12.7	53
W1.0	1:5	50	0.84	¾	125	3480	3050	12.3	53
W1.1			0.93	3	160	3160	2870	9.2	53
W1.2			1.01	6	210	2800	2400	14.3	57
W1.35			1.13	7½	240	2180	1850	15.2	57
X1.0	1:4	50	0.66	½	120	4710	4480	4.7	57
X1.1			0.73	2	140	4230	4100	3.0	56
X1.2			0.80	4	180	3940	3540	10.1	56
X1.35			0.90	8	225	3020	2890	4.3	57
Y1.0	1:5	45	0.77	¾	125	3910	3660	6.3	57
Y1.1			0.84	3	135	3490	3030	13.2	57
Y1.2			0.92	5½	205	3250	2850	12.3	56
Y1.35			1.04	7½	235	2610	2050	21.4	56

TABLE 7—RESULTS OF COMPRESSION TESTS—CONTINUED

Mark	Mix by Volume	Percentage of Sand	Water-Cement Ratio	Slump in in.	Flow Factor	Compressive Strength in lb. per sq. in.		Percentage Loss in Strength for Absorption Specimen	Age in Days of Absorption Specimen when Broken
						Normal at 28 Days	For Absorption Specimen		
Z1.0	1:7	35	0.87	½	120	2600	2230	14.3	58
Z1.1			0.96	1½	135	2330	2080	10.7	55
Z1.2			1.05	4	165	2040	1710	16.2	55
Z1.35			1.19	6	190	1370	1250	8.7	55
a1.0	1:6	35	0.77	½	120	3410	3240	5.2	55
a1.1			0.83	2	130	3230	2890	10.5	61
a1.2			0.93	4½	180	2470	2170	12.2	62
a1.35			1.04	6½	205	1870	1520	18.7	62
b1.0	1:7	30	1.02	¾	120	2160	1950	9.7	53
b1.1			1.12	4	160	1760	1470	16.4	53
b1.2			1.23	6	185	1330	1230	7.5	53
d1.0	1:6	30	0.87	1	120	2640	2320	12.1	52
d1.1			0.96	2	145	2560	2180	14.8	52
d1.2			1.05	5½	185	2070	1600	22.7	52
d1.35			1.18	7½	210	1480	1190	19.6	52
e1.0	1:5	30	0.76	1¼	120	3490	3130	10.2	52
e1.1			0.84	3½	155	3130	2690	14.1	50
e1.2			0.92	6½	205	2480	2040	17.7	50
e1.35			1.04	8	240	1610	1430	11.2	50
f1.0	1:4	30	0.67	1½	130	4470	4060	9.2	50
f1.1			0.73	4½	160	3850	3540	8.0	50
f1.2			0.80	7½	200	3310	2790	15.7	50
f1.35			0.90	8	245	2330	2060	11.6	49
g1.0	1:7	35	1.07	¾	120	1800	1550	13.8	49
g1.1			1.18	2½	140	1590	1370	13.8	55
g1.2			1.29	4½	175	1420	1250	11.9	55
h1.0	1:6	35	0.93	½	120	2320	2120	8.6	55
h1.1			1.02	1½	130	2050	1870	8.8	58
h1.2			1.11	4½	170	1820	1580	13.2	58
h1.35			1.25	6½	225	1470	1210	17.7	58
k1.0	1:5	35	0.80	½	120	2850	2560	10.1	59
k1.1			0.88	2	135	2790	2570	7.8	59
k1.2			0.96	4	165	2570	2220	13.6	58
k1.35			1.08	7½	225	1880	1610	14.3	58

TABLE 8—RESULTS OF FLOW AND ABSORPTION TESTS

Mark	Mix by Volume	Water-cement Ratio	Flow Tests						Percentage of Absorption after				
			Flow in gal. per sq. ft. per hr.		Max. Plus Variation from Aver. 40-50 hr. Flow, per cent	No. of Specimens Tested	Average Age in Days at First Testing	Age in Days at Re-testing	½ hr.	6 hr.	24 hr.	48 hr.	
			20-50 hr. Interval	40-50 hr. Interval									
A1.0	1:7	0.97	0.00119	0.00098	135	5	34	90–100	1.5	3.7	4.5	4.5	⊙
A1.1		1.07	.00179	.00154	95	5	34	100–108	1.7	3.9	4.7	4.8	◬
A1.2		1.18	.00354	.00316	46	4	33	108	1.7	4.2	5.0	5.0	⊡
B1.0	1:6	0.89	0.00070	0.00056	150	5	43	90	1.4	3.4	4.4	4.5	◬
B1.1		0.96	.00087	.00078	54	5	42	100	1.5	3.6	4.6	4.7	⊙
B1.2		1.05	.00173	.00160	150	4	30	91	1.6	4.1	4.9	4.9	⊡
B1.35		1.20	.....	.01190	51	3	38	244	1.8	4.4	5.1	5.1	⊡
C1.0	1:5	0.73	0.00031	0.00022	82	4	35	450	1.3	3.0	4.0	4.2	⊙
C1.1		0.81	.00030	.00021	55	4	37	—	1.4	3.4	4.5	4.5	⊙
C1.2		0.90	.00063	.00056	79	4	48	—	1.4	3.7	4.6	4.7	⊙
C1.35		1.01	.00092	.00080	88	3	36	—	1.6	4.0	4.9	4.9	◬
D1.0	1:4	0.63	0.00022	0.00022	67	5	38	90	1.2	2.8	3.8	4.0	⊙
D1.1		0.66	.00027	.00022	164	5	35	85	1.3	3.0	4.0	4.1	⊙
D1.2		0.74	.00029	.00025	77	5	35	—	1.6	3.6	4.6	4.6	⊙
D1.35		0.81	.00023	.00019	28	5	35	90	1.5	3.6	4.6	4.7	⊙
E1.0	1:7.1	1.02	0.00123	0.00102	18	4	43	—	1.4	3.6	4.5	4.6	◬
E1.1		1.09	.00152	.00116	81	4	42	—	1.6	3.8	4.8	4.8	⊡
E1.2		1.15	.00280	.00232	77	4	34	90	1.6	4.1	5.0	5.0	⊡
F1.0	1:6	0.85	0.00049	0.00042	105	5	41	—	1.3	3.3	4.2	4.3	⊙
F1.1		0.95	.00083	.00068	32	5	41	103	1.4	3.7	4.7	4.7	⊙
F1.2		1.04	.00086	.00068	47	5	44	93	1.6	4.0	4.9	5.0	⊙
F1.35		1.17	.00233	.00218	88	4	50	85	1.7	4.4	5.3	5.3	◬
G1.0	1:5	0.71	0.00041	0.00035	71	4	48	103	1.3	3.1	4.1	4.2	⊙
G1.1		0.78	.00020	.00013	41	5	35	—	1.4	3.4	4.4	4.5	⊙
G1.2		0.85	.00029	.00022	139	4	41	—	1.5	3.7	4.8	4.8	⊙
G1.35		0.97	.00047	.00035	43	4	35	—	1.6	4.2	5.2	5.3	⊙
H1.0	1:5.6	1.00	0.00139	0.00122	39	5	41	—	1.5	4.2	5.4	5.5	◬
H1.1		1.10	.00217	.00205	70	4	44	—	1.6	4.9	5.9	5.9	◬
H1.2		1.21	.....	.00500	72	5	42	—	1.9	4.9	6.0	6.0	⊡
I1.0	1:4.7	0.84	0.00077	0.00070	114	5	36	—	1.4	3.9	5.1	5.2	◬
I1.1		0.93	.00066	.00058	176	5	36	110	1.6	4.1	5.2	5.3	◬
I1.2		1.01	.00151	.00138	125	5	43	—	1.8	4.5	5.7	5.8	◬
I1.35		1.15	.00196	.00170	71	4	32	—	1.8	4.8	6.0	6.1	◬
J1.0	1:4	0.72	0.00032	0.00028	9	4	43	—	1.2	3.2	4.4	4.6	⊙
J1.1		0.79	.00033	.00020	43	4	43	—	1.4	3.6	4.8	4.9	⊙
J1.2		0.86	.00044	.00035	14	4	43	—	1.5	3.9	5.0	5.1	⊙
J1.35		0.97	.00057	.00045	33	4	44	—	1.7	4.3	5.5	5.6	⊙
K1.0	1:6	0.74	0.00049	0.00040	50	4	40	86	1.3	3.2	4.1	4.2	⊙
K1.08		0.80	.00034	.00023	74	4	46	—	1.3	3.2	4.2	4.3	⊙
K1.16		0.86	.00059	.00050	40	4	41	—	1.4	3.4	4.4	4.5	⊙
K1.3		0.97	.00081	.00070	57	4	40	85	1.5	3.6	4.6	4.7	⊙
L1.0	1:5	0.74	0.00035	0.00026	115	5	44	—	1.2	3.1	4.2	4.3	⊙
L1.1		0.82	.00059	.00052	73	5	39	—	1.2	3.2	4.3	4.5	⊙
L1.2		0.89	.00059	.00042	90	5	38	85	1.3	3.5	4.7	4.8	⊙
L1.35		1.01	.00127	.00096	98	5	44	—	1.4	3.8	5.0	5.1	◬
M1.0	1:6	0.88	0.00060	0.00046	30	5	39	—	1.3	3.4	4.4	4.6	⊙
M1.1		0.97	.00064	.00056	61	5	42	—	1.2	3.3	4.6	4.6	◬
M1.2		1.07	.00136	.00122	64	5	40	—	1.4	3.7	4.8	4.9	◬
M1.35		1.20	.00574	.00520	54	4	38	—	1.6	4.2	5.2	5.3	⊡

TABLE 8—RESULTS OF FLOW AND ABSORPTION TESTS—CONTINUED

Mark	Mix by Volume	Water-cement Ratio	Flow Tests: Flow in gal. per sq. ft. per hr. 20-50 hr. Interval	Flow Tests: Flow in gal. per sq. ft. per hr. 40-50 hr. Interval	Flow Tests: Max. Plus Variation from Aver. 40-50 hr. Flow, per cent	Flow Tests: No. of Specimens Tested	Flow Tests: Average Age in Days at First Testing	Flow Tests: Age in Days at Re-testing	Percentage of Absorption after ½ hr.	Percentage of Absorption after 6 hr.	Percentage of Absorption after 24 hr.	Percentage of Absorption after 48 hr.	
N1.0	1:5	0.74	0.00041	0.00034	18	5	39	380	1.2	3.1	4.0	4.1	⊙
N1.1		0.82	.00042	.00038	38	4	39	—	1.4	3.5	4.4	4.4	⊙
N1.2		0.90	.00066	.00052	92	5	41	95–480	1.5	3.8	4.8	4.8	⊙
N1.35		1.02	.00087	.00075	45	4	38	—	1.7	4.2	5.2	5.3	△
O1.0	1:7	0.98	0.....	0.00216	48	5	46	130–300	1.5	3.8	4.8	4.8	△
O1.1		1.09	.00433	.00408	72	5	42	90–150	1.7	4.2	5.0	5.1	□
O1.2		1.19	.00716	.00664	10	3	41	—	1.7	4.5	5.3	5.3	□
P1.0	1:6	0.89	0.00080	0.00058	55	5	36	—	1.4	3.4	4.5	4.5	△
P1.1		0.99	.00084	.00070	114	5	37	—	1.3	3.7	4.9	5.0	△
P1.2		1.09	.00127	.00105	60	4	41	—	1.4	4.0	5.3	5.4	△
P1.35		1.23	.00599	.00536	40	4	43	—	1.6	4.5	5.5	5.6	□
Q1.0	1:5	0.74	0.00039	0.00032	23	4	38	—	1.2	3.0	4.1	4.2	⊙
Q1.1		0.80	.00038	.00032	25	5	37	—	1.3	3.3	4.6	4.7	⊙
Q1.2		0.90	.00057	.00046	52	5	39	92	1.5	3.8	5.1	5.1	⊙
Q1.35		1.02	.00115	.00100	20	4	47	285	1.6	4.1	5.4	5.5	△
R1.0	1:4	0.64	0.00033	0.00028	135	5	37	—	1.2	2.8	3.8	4.0	⊙
R1.1		0.70	.00030	.00027	106	5	39	—	1.4	3.2	4.2	4.4	⊙
R1.2		0.77	.00093	.00070	100	5	39	77	1.4	3.5	4.7	4.8	⊙
R1.35		0.88	.00191	.00138	103	5	39	81–423	1.7	4.0	5.2	5.3	△
S1.0	1:6	0.92	0.00159	0.00132	67	5	38	—	1.4	3.6	4.8	4.9	△
S1.1		1.02	.00267	.00238	51	5	39	283	1.6	3.9	5.2	5.3	□
S1.2		1.12	.00229	.00190	63	5	38	330	1.6	4.4	5.7	5.7	□
S1.35		1.27	.01350	.01230	48	3	41	—	2.0	5.1	6.0	6.1	□
T1.0	1:5	0.79	0.00041	0.00034	47	5	38	—	1.3	3.4	4.7	4.9	⊙
T1.1		0.88	.00084	.00072	39	5	40	75	1.4	3.9	4.9	5.0	△
T1.2		0.96	.00094	.00072	67	5	40	75	1.5	4.1	5.2	5.4	△
T1.35		1.09	.00141	.00128	88	4	41	—	1.8	4.8	5.8	5.9	△
U1.0	1:4	0.69	0.00016	0.00008	150	4	41	—	1.4	3.4	4.4	4.5	⊙
U1.1		0.76	.00031	.00023	21	5	39	—	1.5	3.8	4.8	5.0	⊙
U1.2		0.84	.00049	.00042	67	5	40	—	1.5	3.9	4.9	4.9	⊙
U1.35		0.94	.00059	.00052	35	5	38	—	1.8	4.6	5.5	5.6	⊙
V1.0	1:6	1.02	0.00139	0.00115	124	4	39	—	1.5	4.2	5.2	5.3	△
V1.1		1.12	.00214	.00185	72	4	41	—	1.6	4.4	5.5	5.6	△
V1.2		1.23	.....	.00625	39	4	41	—	1.8	4.6	5.6	5.8	□
V1.35		1.38	.02600	.02430	48	3	45	115	2.0	5.0	6.0	6.1	□
W1.0	1:5	0.84	0.....	0.00070	71	4	41	—	1.2	3.4	4.7	4.8	⊙
W1.1		0.93	.00074	.00058	72	4	39	—	1.5	3.7	4.8	4.9	⊙
W1.2		1.01	.00141	.00116	81	5	38	—	1.5	4.0	5.3	5.4	△
W1.35		1.13	.00160	.00142	97	4	38	—	1.6	4.4	5.7	5.8	△
X1.0	1:4	0.66	0.00069	0.00056	61	5	40	—	1.2	3.0	4.2	4.3	⊙
X1.1		0.73	.00061	.00052	131	5	40	120	1.2	3.2	4.3	4.4	⊙
X1.2		0.80	.00059	.00048	150	5	40	120	1.3	3.6	4.7	4.8	⊙
X1.35		0.90	.00073	.00062	158	5	39	—	1.6	4.3	5.3	5.4	△
Y1.0	1:5	0.77	0.00105	0.00092	74	5	46	—	1.2	3.2	4.3	4.4	△
Y1.1		0.84	.00065	.00048	67	5	40	—	1.4	3.7	4.7	4.8	⊙
Y1.2		0.92	.00078	.00064	88	5	38	—	1.4	3.7	4.8	5.0	△
Y1.35		1.04	.00122	.00106	60	5	48	330	1.6	4.2	5.4	5.5	△

TABLE 8—RESULTS OF FLOW AND ABSORPTION TESTS—CONTINUED

Mark	Mix by Volume	Water-Cement Ratio	Flow Tests: Flow in gal. per sq. ft. per hr. 20-50 hr. Interval	Flow Tests: Flow in gal. per sq. ft. per hr. 40-50 hr. Interval	Flow Tests: Max. Plus Variation from Aver. 40-50 hr. Flow, per cent	Flow Tests: No. of Specimens Tested	Flow Tests: Average Age in Days at First Testing	Flow Tests: Age in Days at Re-testing	Percentage of Absorption after ½ hr.	6 hr.	24 hr.	48 hr.	
Z1.0	1:7	0.87	0.00153	0.00122	23	4	39	—	1.3	3.2	4.0	4.2	◬
Z1.1		0.96	.00222	.00186	223	5	40	120	1.2	3.3	4.4	4.4	◬
Z1.2		1.05	.00239	.00200	130	4	44	120	1.3	3.5	4.5	4.6	◬
Z1.35		1.19	.00565	.00522	72	4	50	—	1.4	3.7	4.6	4.7	⊡
a1.0	1:6	0.77	0.00088	0.00072	67	5	43	—	1.1	2.8	3.9	4.1	◬
a1.1		0.83	.00115	.00086	51	5	44	—	1.3	3.3	4.3	4.3	◬
a1.2		0.93	.00082	.00058	38	4	47	—	1.4	3.5	4.4	4.4	⊙
a1.35		1.04	.00157	.00140	50	4	46	110	1.5	3.7	4.6	4.7	◬
b1.0	1:7	1.02	0.00189	0.00162	172	4	37	—	1.2	3.4	4.3	4.5	◬
b1.1		1.12	.00407	.00358	37	4	40	—	1.4	3.9	4.6	4.8	⊡
b1.2		1.23	.....	.00436	47	5	44	—	1.5	4.2	4.8	4.8	⊡
d1.0	1:6	0.87	0.00106	0.00084	43	5	36	—	1.3	3.4	4.2	4.2	◬
d1.1		0.96	.00145	.00118	95	4	37	—	1.4	3.6	4.5	4.6	◬
d1.2		1.05	.00166	.00140	29	4	35	100	1.5	4.0	4.8	4.9	◬
d1.35		1.18	.00552	.00470	143	5	41	188–450	1.6	4.2	4.9	4.9	⊡
e1.0	1:5	0.76	0.00042	0.00030	133	5	36	—	1.3	3.2	4.2	4.2	⊙
e1.1		0.84	.00051	.00044	127	5	37	—	1.3	3.3	4.3	4.4	⊙
e1.2		0.92	.00069	.00050	80	5	37	105	1.4	3.6	4.5	4.5	⊙
e1.35		1.04	.00164	.00138	38	4	41	—	1.6	4.0	4.8	4.8	◬
f1.0	1:4	0.67	0.00021	0.00015	62	5	39	100	1.3	3.0	3.9	4.0	◌́
f1.1		0.73	.00019	.00012	61	4	37	—	1.3	3.2	4.2	4.3	⊙
f1.2		0.80	.00029	.00018	47	4	37	—	1.4	3.4	4.4	4.5	⊙
f1.35		0.90	.00079	.00066	52	5	36	215	1.5	3.8	4.8	4.8	◬
g1.0	1:7	1.07	0.00346	0.00310	94	4	35	196	1.5	3.9	4.8	4.8	⊡
g1.1		1.18	.00579	.00546	71	5	45	—	1.6	4.4	5.2	5.2	⊡
g1.2		1.29	.00719	.00658	70	4	38	85	1.7	4.6	5.3	5.3	⊡
h1.0	1:6	0.93	0.00265	0.00212	65	5	37	—	1.4	3.8	4.8	4.9	◬
h1.1		1.02	.00232	.00188	92	5	46	90	1.6	4.4	5.1	5.2	◬
h1.2		1.11	.00346	.00316	58	5	37	—	1.6	4.5	5.3	5.3	⊡
h1.35		1.25	.00965	.00902	38	4	43	—	1.9	4.9	5.4	5.4	⊡
k1.0	1:5	0.80	0.00075	0.00062	61	5	36	—	1.3	3.6	4.7	4.8	◌́
k1.1		0.88	.00059	.00042	90	5	36	—	1.5	4.0	5.1	5.1	⊙
k1.2		0.96	.00053	.00040	25	5	36	—	1.5	4.0	4.9	4.9	⊙
k1.35		1.08	.00134	.00116	98	5	37	95	1.7	4.4	5.2	5.3	◬

Addenda for Table 8.

Column 8—"Average Age in Days at First Testing" includes age of rerun specimens when age at retest was less than 75 days.

Column 9—"Age in Days at Retesting" includes age of rerun specimens when more than 75 days old.

Explanation of Symbols.

◌́ Surface dry, but greater flow than wetter mix.

⊙ Surface dry.

◬ Surface somewhat wet.

⊡ Surface quite wet.

TABLE 9—DENSITIES, CEMENT-VOIDS RATIOS AND OTHER PHYSICAL CONSTANTS FOR MIXES

Mark	Mix by Volume	Water-Cement Ratio	Density	Absolute Vol. of Cement per Unit Vol. of Concrete	Cement-Voids Ratio	Cement-Space Ratio	Cement-Dry Agg. Ratio by Weight
A1.0	1:7	0.97	0.844	0.0719	0.464	0.316	0.108
A1.1		1.07	.834	.0711	.429	.300	
A1.2		1.18	.825	.0703	.401	.286	
B1.0	1:6	0.89	0.837	0.0817	0.503	0.334	0.126
B1.1		0.96	.833	.0814	.489	.328	
B1.2		1.05	.823	.0804	.453	.312	
B1.35		1.20	.816	.0797	.434	.302	
C1.0	1:5	0.73	0.836	0.0963	0.590	0.370	0.151
C1.1		0.81	.828	.0953	.556	.357	
C1.2		0.90	.823	.0948	.537	.349	
C1.35		1.01	.818	.0941	.518	.341	
D1.0	1:4	0.63	0.838	0.1175	0.728	0.421	0.190
D1.1		0.66	.833	.1167	.697	.411	
D1.2		0.74	.822	.1152	.647	.393	
D1.35		0.81	.819	.1147	.632	.387	
E1.0	1:7.1	1.02	0.829	0.0698	0.408	0.290	0.107
E1.1		1.09	.828	.0697	.405	.288	
E1.2		1.15	.830	.0698	.411	.291	
F1.0	1:6	0.85	0.840	0.0822	0.515	0.340	0.126
F1.1		0.95	.835	.0817	.496	.331	
F1.2		1.04	.828	.0810	.471	.320	
F1.35		1.17	.821	.0804	.450	.310	
G1.0	1:5	0.71	0.839	0.0964	0.598	0.374	0.151
G1.1		0.78	.835	.0960	.581	.367	
G1.2		0.85	.825	.0949	.543	.352	
G1.35		0.97	.817	.0939	.514	.339	
H1.0	1:5.6	1.00	0.810	0.0791	0.419	0.294	0.126
H1.1		1.10	.801	.0782	.394	.282	
H1.2		1.21	.795	.0777	.382	.275	
I1.0	1:4.7	0.84	0.812	0.0933	0.497	0.331	0.151
I1.1		0.93	.810	.0931	.490	.328	
I1.2		1.01	.800	.0919	.460	.314	
I1.35		1.15	.797	.0916	.451	.311	
J1.0	1:4	0.72	0.818	0.1078	0.595	0.372	0.177
J1.1		0.79	.807	.1064	.552	.355	
J1.2		0.86	.803	.1060	.539	.350	
J1.35		0.97	.794	.1047	.510	.337	
K1.0	1:6	0.74	0.850	0.0800	0.536	0.349	0.121
K1.08		0.80	.849	.0798	.528	.345	
K1.16		0.86	.842	.0792	.501	.334	
K1.3		0.97	.841	.0791	.497	.332	
L1.0	1:5	0.74	0.847	0.0937	0.611	0.379	0.145
L1.1		0.82	.839	.0929	.579	.366	
L1.2		0.89	.830	.0919	.542	.351	
L1.35		1.01	.820	.0908	.506	.336	
M1.0	1:6	0.88	0.835	0.0787	0.476	0.322	0.121
M1.1		0.97	.831	.0785	.465	.317	
M1.2		1.07	.824	.0777	.441	.306	
M1.35		1.20	.816	.0770	.420	.296	

TABLE 9—DENSITIES, CEMENT-VOIDS RATIOS AND OTHER PHYSICAL CONSTANTS FOR MIXES—CONTINUED

Mark	Mix by Volume	Water-Cement Ratio	Density	Absolute Vol. of Cement per Unit Vol. of Concrete	Cement-Voids Ratio	Cement-Space Ratio	Cement-Dry Agg. Ratio by Weight
N1.0	1:5	0.74	0.836	0.0929	0.565	0.361	0.147
N1.1		0.82	.831	.0924	.548	.354	
N1.2		0.90	.826	.0918	.529	.345	
N1.35		1.02	.821	.0912	.509	.337	
O1.0	1:7	0.98	0.842	0.0694	0.439	0.305	0.105
O1.1		1.09	.834	.0687	.414	.293	
O1.2		1.19	.822	.0678	.381	.276	
P1.0	1:6	0.89	0.837	0.0791	0.484	0.326	0.122
P1.1		0.99	.829	.0784	.460	.315	
P1.2		1.09	.821	.0777	.435	.303	
P1.35		1.23	.812	.0768	.409	.290	
Q1.0	1:5	0.74	0.837	0.0937	0.576	0.365	0.147
Q1.1		0.80	.834	.0934	.564	.360	
Q1.2		0.90	.818	.0916	.503	.334	
Q1.35		1.02	.812	.0909	.483	.326	
R1.0	1:4	0.64	0.834	0.1138	0.689	0.407	0.185
R1.1		0.70	.828	.1130	.660	.397	
R1.2		0.77	.818	.1116	.615	.380	
R1.35		0.88	.805	.1098	.564	.360	
S1.0	1:6	0.92	0.827	0.0782	0.452	0.311	0.122
S1.1		1.02	.822	.0777	.438	.304	
S1.2		1.12	.813	.0769	.411	.291	
S1.35		1.27	.800	.0756	.377	.274	
T1.0	1:5	0.79	0.828	0.0921	0.534	0.348	0.146
T1.1		0.88	.823	.0915	.517	.340	
T1.2		0.96	.813	.0904	.483	.326	
T1.35		1.09	.803	.0895	.455	.312	
U1.0	1:4	0.69	0.827	0.1114	0.644	0.391	0.182
U1.1		0.76	.815	.1099	.595	.373	
U1.2		0.84	.811	.1093	.579	.366	
U1.35		0.94	.798	.1076	.533	.347	
V1.0	1:6	1.02	0.810	0.0774	0.407	0.289	0.123
V1.1		1.12	.805	.0768	.395	.283	
V1.2		1.23	.797	.0760	.374	.272	
V1.35		1.38	.789	.0754	.358	.263	
W1.0	1:5	0.84	0.814	0.0913	0.493	0.330	0.148
W1.1		0.93	.811	.0906	.479	.324	
W1.2		1.01	.805	.0903	.464	.316	
W1.35		1.13	.799	.0896	.446	.308	
X1.0	1:4	0.66	0.823	0.1123	0.635	0.388	0.184
X1.1		0.73	.816	.1113	.607	.377	
X1.2		0.80	.810	.1105	.584	.368	
X1.35		0.90	.801	.1093	.571	.355	
Y1.0	1:5	0.77	0.821	0.0960	0.536	0.349	0.155
Y1.1		0.84	.816	.0955	.517	.341	
Y1.2		0.92	.808	.0946	.492	.330	
Y1.35		1.04	.801	.0938	.474	.321	
Z1.0	1:7	0.87	0.853	0.0685	0.468	0.319	0.102
Z1.1		0.96	.852	.0685	.464	.316	
Z1.2		1.05	.850	.0683	.458	.313	
Z1.35		1.19	.842	.0678	.430	.300	

TABLE 9—DENSITIES, CEMENT-VOIDS RATIOS AND OTHER PHYSICAL CONSTANTS FOR MIXES—CONTINUED

Mark	Mix by Volume	Water-Cement Ratio	Density	Absolute Vol. of Cement per Unit Vol. of Concrete	Cement-Voids Ratio	Cement-Space Ratio	Cement-Dry Agg. Ratio by Weight
a1.0	1:6	0.77	0.855	0.0796	0.551	0.355	0.120
a1.1		0.83	.847	.0788	.516	.340	
a1.2		0.93	.841	.0782	.491	.329	
a1.35		1.04	.834	.0776	.469	.319	
b1.0	1:7	1.02	0.843	0.0667	0.425	0.298	0.101
b1.1		1.12	.836	.0662	.405	.288	
b1.2		1.23	.831	.0657	.390	.280	
d1.0	1:6	0.87	0.842	0.0763	0.485	0.326	0.117
d1.1		0.96	.843	.0765	.489	.328	
d1.2		1.05	.835	.0757	.459	.314	
d1.35		1.18	.830	.0752	.443	.306	
e1.0	1:5	0.76	0.844	0.0900	0.578	0.366	0.140
e1.1		0.84	.840	.0896	.561	.359	
e1.2		0.92	.836	.0892	.545	.352	
e1.35		1.04	.825	.0880	.502	.334	
f1.0	1:4	0.67	0.840	0.1094	0.684	0.406	0.176
f1.1		0.73	.833	.1086	.651	.394	
f1.2		0.80	.826	.1075	.619	.382	
f1.35		0.90	.813	.1060	.568	.362	
g1.0	1:7	1.07	0.834	0.0650	0.392	0.281	0.099
g1.1		1.18	.831	.0648	.383	.276	
g1.2		1.29	.825	.0643	.367	.268	
h1.0	1:6	0.93	0.839	0.0751	0.467	0.318	0.115
h1.1		1.02	.834	.0746	.449	.310	
h1.2		1.11	.836	.0748	.458	.313	
h1.35		1.25	.822	.0735	.415	.292	
k1.0	1:5	0.80	0.839	0.0882	0.550	0.353	0.138
k1.1		0.88	.836	.0879	.537	.349	
k1.2		0.96	.828	.0872	.509	.336	
k1.35		1.08	.822	.0865	.487	.327	

TABLE 10—RESULTS OF FLOW TESTS AT 100 LB. PER SQ. IN. PRESSURE

Mark	Flow in gal. per sq. ft. per hr.		Maximum Plus Variation from Average Flow in per cent		No. of Spec. Tested	Age in months	Condition of Surface
	40-50 hr. Interval	100-110 hr. Interval	40-50 hr. Interval	100-110 hr. Interval			
D1.0	0.000325	0.00016	54	128	3	11	dry
D1.35	.00070	.00045	114	167	5	16	dry
F1.1	.00022	.000175	127	71	4	12	moist
G1.1	.00030	.000175	100	69	5	10	moist
H1.0	.00185	.00140	130	115	5	11	moist
L1.0	.00040	.00020	20	50	5	15	dry
L1.1	.00104	.00060	83	66	5	14	dry
P1.0	.00080	.00060	60	58	5	14	moist
Q1.0	.00050	.00025	100	...	5	14	dry
R1.0	.00032	.00010	50	54	5	14	dry
R1.35	.000184	.00017	180	114	4	15	moist
S1.2	.00480	.00400	73	120	5	14	wet
T1.1	.00090	.00065	66	85	5	14	moist
U1.2	.00075	.00050	48	68	5	14	moist
W1.2	.00212	.00112	53	145	5	14	moist
X1.0	.00092	.00050	47	60	5	15	moist
X1.35	.00110	.00045	63	189	5	15	dry
Y1.2	.00100	.00049	75	125	5	15	moist
Y1.35	.00200	.00140	112	50	5	15	moist

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F. R. McMillan. *Basic Principles of Concrete Making.*

Readers are referred to the JOURNAL *for November, 1931 (V. 28), for discussion which may develop. Such discussion should reach the Secretary by Oct. 1, 1931.*

Discussion of three Papers under the general heading:*

Workability and Admixtures

"A Study of the Flow Table and the Slump Test"
BY GEORGE A. SMITH AND SANFORD W. BENHAM

"A Study of Slump and Flow of Concrete"
BY INGE LYSE AND W. R. JOHNSON

"Admixtures and Workability of Concrete"
BY G. M. WILLIAMS

The discussion here presented based on three closely related and previously published papers was led by R. B. Young and J. C. Pearson, selected by the Program committee to present convention discussion, in an effort at some reconciliation of view of a moot subject. The impromptu convention discussion; the written discussion prior and subsequent to the convention, is so voluminous as to exceed available space in this Journal. *It is therefore continued to the* Journal *for November, 1931, (Vol. 28) and the publication of several interesting contributions already available is necessarily deferred.*—Editor

BY R. B. YOUNG

I understand that I was chosen to introduce this discussion on the basis that I am a disinterested party, which might be interpreted to mean that I do not know anything about it and therefore am a safe speaker. After reading the papers, I am less prejudiced than I was before because I know less about the subject than I did before. The papers are conflicting and I have been unable to comply with the specification and bring them together. The papers by Mr. Smith and Mr. Benham, and Mr. Lyse and Mr. Johnson, both report experimental comparisons of the flow table and slump as measures of workability and con-

*The three papers were discussed at the 27th Annual Convention, Milwaukee, February 24-26, subsequent to publication in this Journal: "A Study of the Flow Table and the Slump Test," by George A. Smith and Sanford W. Benham, January, 1931, Vol. 27, p. 420. "A Study of Slump and Flow of Concrete," by Inge Lyse and W. R. Johnson, January, 1931, V. 27, p. 439. "Admixtures and Workability of Concrete," by G. M. Williams, February, 1931, Vol. 27, p. 647.

sistency, using similar materials and proportions but different methods of approach. They have arrived at almost opposite conclusions as far as the main point is concerned. I have spent some time trying to bring these two papers together and I have not been very successful, because the fact remains that they record—and I am unable to find a reason why—such different conclusions. It seems to me, after studying the methods and test data obtained, that the data are unconvincing and the reasonable conclusion is that neither is the measure of the workability. Workability, as I believe most men handling concrete will agree, is that combination of the properties of concrete which allow us to handle and place it with reasonable labor and minimum segregation. Neither the slump test nor the flow test measures this property. The slump test has been used as a control test in the field for a great many years, and its widespread adoption is proof of its merit for the purpose. Smith and Benham point out that the slump test acts differently with different types of mixtures. They state this: "During the making of the tests four characteristic conditions as to general shapes of the resulting slump specimen were observed. First; specimens slumped only small amounts and retained somewhat the same shape as the molded sample. Second; specimens slumped several inches but held together, bulging out uniformly as the slump took place." That is the ideal condition when it is used as a control work. "Third; a portion of the specimen stood up on one side and the remainder fell away; not a usual condition and always checked by a second trial. Fourth; top slushed down, leaving a mound, generally of compacted aggregate in the center." That is, the concrete had begun to segregate. The observations of this paper have been quite the general experience of most users of this test. The principal value of the slump test is its use with a certain class of concrete where a uniform degree of workability is represented by a given slump. With fixed materials and a fixed combination of those materials, you can represent the workability that you are after by some slump and later you can be fairly certain that the concrete from which the sample was taken has a workability close to what you are after, if it has that same slump, but if you change the conditions, using other materials possibly, it will take another degree of slump to represent the same degree

of workability. As a control test, that idiosyncrasy of the slump is not so important. However, when it comes to measuring workability and the effect of admixtures, those little peculiarities make it difficult to interpret the result.

The flow test is almost entirely a laboratory test and is not subject to the same faults as the slump test, but is usable over a greater range of mixtures. Its principal value has been to define the approximate workability of a mix in laboratory work. There again I would say that if you find in one of these discussions, that the investigator has had a flow of 80, another laboratory worker has a good idea of what that mixture looks like, but beyond giving the other investigator an idea of what the concrete looks like, I think the test has little practical use. It is entirely too cumbersome for a practical field test.

To increase the range of mixtures, the investigators reporting in these two papers have introduced admixtures into their tests. While the purpose was to increase the range of mixtures studied, I think that one of the most likely results will be to add a good deal of discussion to the papers. In fairness to the authors, I think it should be pointed out that none of them is presenting his data to make a case for or against admixtures. These two papers present studies at workability and tests of workability, and, as I read the papers, the introduction of admixtures was more or less incidental. I think that neither the slump test nor the flow test can be safely used to determine the value of admixtures in concrete, and I, for one, would not accept any conclusions based thereon.

The paper by Professor Williams presents a new method of approach to the study of workability and the value of admixtures. In submitting his data, he made the following comments, which are worth reading. They give a little different slant to his subsequent discussion, if his viewpoint is taken into account. The author writes:

> My purpose was to outline a method of test for the rating of admixtures rather than to discuss the merits or demerits of any particular one or group. A satisfactory test should be established first. Had I intended to compare the admixtures in this manner, I would have included several others which have been used commercially, one of them approaching fairly near to No. 6 in efficiency. I included a minimum of test data in tabular form to make com-

parisons easier. Strength, porosity and permeability must be taken into account in any final rating of an admixture, but I felt that to discuss these factors at length would only divert attention from the test methods proposed, toward the admixtures themselves. The statement that admixtures "do not appreciably affect strength or permeability when used in the amount or proportion specified by the manufacturer" briefly sums up any discussion which I might make in connection with these factors.

Professor Williams discusses in his paper certain fundamental facts. He compares cement and admixtures as to the volume of paste given when equal weights are mixed to give pastes of equal flow; also, the effect on the volume of mortar under the same conditions and the effect on preventing segregation when jigged, and then proceeds to demonstrate the reasonableness of his method of study by producing a series of concretes which were alike in their tendency to segregate when made up as suggested by his previous tests. In closing his paper, he takes a very sane view of the possible uses and abuses of admixtures, which I will quote in closing:

While admixtures are relatively more efficient than cement in obtaining a concrete of workability suited to any given condition of transportation and placement, the writer does not suggest that their use is always necessary or advisable. For concrete of any required cement content first attention should be directed to the most desirable sand-gravel ratio. Relative increase in sand content increases mortar volume which in turn permits the use of a less fluid mortar, thereby increasing workability and postponing likelihood of segregation. After adjusting the sand-gravel ratio and finding segregation under the conditions of use, or that the mix is near the segregation limit the use of an admixture will be advantageous. In work where a surface as true as the face of the mold is desired or in any case where the available fine aggregate is coarse and harsh, lacking in fine particles, the use of an admixture may be of value in obtaining satisfactory results.

The factors of strength and watertightness should not be forgotten. Generally these properties cannot be sacrificed to obtain workability. Tests which the writer has made quite definitely indicate that admixtures do not appreciably affect strength or permeability when used in the amounts or proportions specified by the manufacturer. In cases where excess quantities of an admixture are needed to produce satisfactory workability the possibility of reduction in strength and watertightness must receive consideration. On the basis of workability alone, the data in Table 1 would indicate the use of an admixture to be dependent mainly upon the relative cost of cements and admixtures. The use of admixtures 2 to 5 would not be economical unless their cost were less than twice, and No. 6 less than five or six times, the cost of portland cement. Consideration should also be given to the fact that cement not only increases workability as an admixture but at the same time it definitely increases strength and impermeability.

BY J. C. PEARSON

These valuable papers on slump and flow tests ought to be considered about the last word on the subject. They represent a very large amount of careful and painstaking work—how large can only be adequately realized by those who have studied the data closely. There are many interesting details worthy of attention, but this writer begs the privilege of departing from the usual custom, and of confining himself to a discussion of the present status of the workability problem and of the possibilities that are suggested by the data in the papers, and by certain experiments which have been carried out in the Research Laboratory of the Lehigh Portland Cement Co.

The outstanding feature of the papers by Smith-Benham and Lyse-Johnson is the demonstration that for concretes of different composition the ratio of slump to flow as measured on the flow-table is not constant, but increases with the richness of the mixtures. The significance of this important fact can be brought out on certain assumptions, if we consider briefly the characteristics of plastic materials.

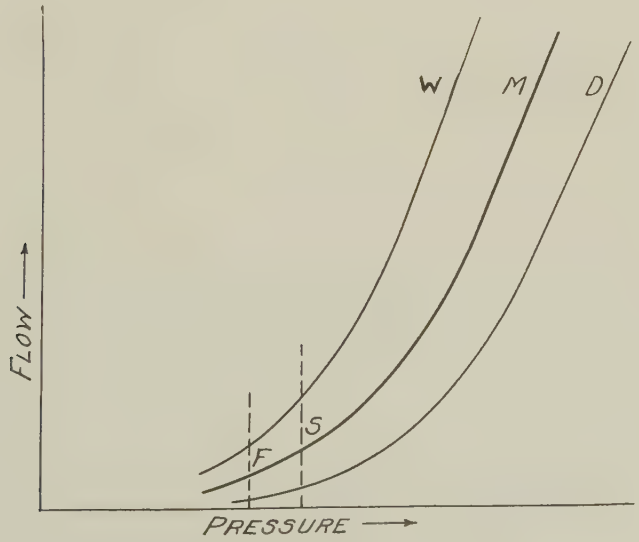
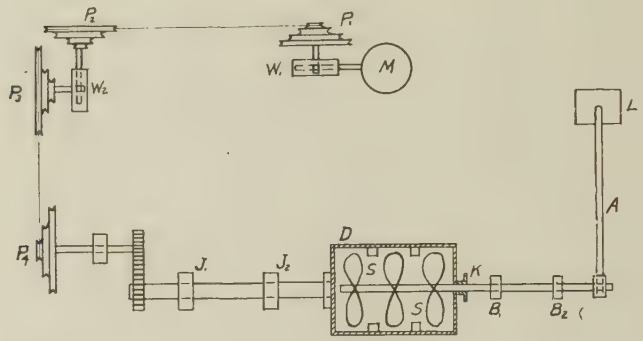


FIG. 1



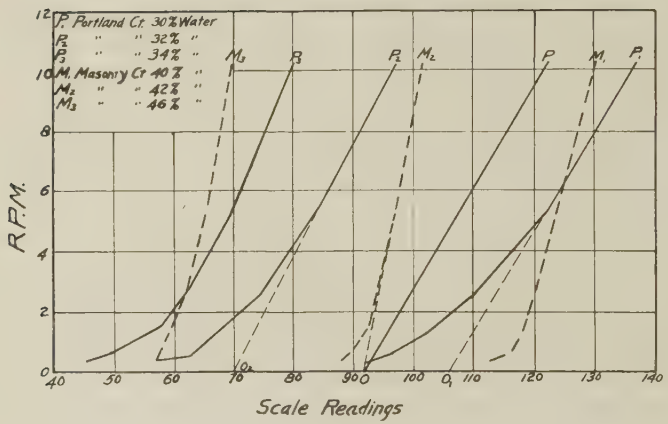


FIG. 2

If a mass of freshly mixed concrete could be forced out of a container through an orifice or a tube in such manner that the rate of shearing flow could be determined under different pressures we should expect to find the flow-pressure relations represented by a curve like one of those in Fig. 1. Confining ourselves for the moment to the heavy curve, M, we see that at first, for slight or moderate pressures, there would be no flow at all; at somewhat higher pressures the flow would be sluggish, but increasing with increase in pressure; and finally after the pressure had reached a certain value the relation of flow to pressure would become linear, or approximately so. I am not aware that an experiment of this

kind has ever been carried out successfully with concrete, but the relation is a typical one for plastic materials, and we have quite a lot of data substantiating the assumption that the relation holds for concrete.[1] The slope of the straight-line portion of the curve is commonly referred to as the "mobility," and we have found from a study of cement pastes that this factor of mobility is the one which seems most nearly allied to, and best adapted for, a comparative plasticity rating.

The upper portion of Fig. 2 illustrates a device used in our laboratory for studying the flow-force relations in cement pastes. A mixer drum, D, is mounted on the end of a stout shaft supported in the bearings J_1, J_2, and is driven by means of gears, belts, step pulleys, reducing units and a small constant speed motor, M. By simply changing the position of the belts the drum may be driven at 12 different speeds. Projecting into the drum through a low-friction packing gland, K, is another shaft supported in ball bearings, B_1, B_2, and blades or paddles, S, of different types, may be mounted on the shaft inside the drum. The outer end of this shaft carries a rigidly clamped arm, A, the far end of which rests on a platform balance, L. When the drum is filled with paste and put in rotation the blades, S, shear through the paste, and a certain resistance is set up by them which is indicated on the balance, L. Thus we have a system for measuring flow-force relations in which the speed of the drum is analagous to the rate of flow, and the reacting torque of the stator blades is the force.

Some typical results obtained with this device are shown in the lower portion of Fig. 2. The curves P_1, P_2, and P_3 were obtained from portland cement pastes containing different amounts of water; the curves M_1, M_2 and M_3 were obtained from masonry cement pastes containing different amounts of water. What I wish to call attention to particularly in this figure is the fact that change in the water content of the pastes shifts the curves to the right or left on the diagram with only a slight change in their slopes. This slight change in slope enables the interpolation of a curve with an intermediate slope, such as the line OP, which may be compared directly with the slope of the line OM_2, having the same intercept on the pressure axis. Now we believe that when

[1]Purrington & Loring obtained curves of this general type for concrete by measuring the power input and speed of a mixer. See A. S. T. M. Proceedings Vol. 30, Pt. II, p. 654-667, 1930.

two pastes are compared in this manner, the one which gives the greater slope, that is, the one which has the higher mobility, is the more plastic. The physical significance of this is that if one paste is more plastic than another, it shears more easily and flows more readily than the other when it is mixed in a pan or spread with a trowel. Furthermore, experience indicates that this property of the paste is carried over into mortars and concretes, and the degree of it that manifests itself depends upon the quantity of the paste in proportion to the aggregates, and also to a greater or less extent upon other factors, such as the grading of the aggregates, the presence of admixtures, etc.

This presentation is already familiar to some of you who are members of A. S. T. M. Committee C-9, but I have gone into the matter of these paste tests in some detail because of its direct bearing on the question under discussion. Returning now to Fig. 1, we may show why the flow and slump tests function as they do, under certain assumptions. The effective force that produces flow in a mass of concrete on a flow table is small, corresponding to a low pressure value on the diagram. The point F on the typical curve M may be taken to represent the region in which the flow table operates, corresponding to a relatively small deformation in the concrete. In the slump test the effective force tending to cause subsidence of the cone corresponds to a somewhat higher value on the diagram, represented, let us say, by the abscissa of the point S. Now in a given mixture, as shown above for pastes, moderate change in water content would be expected to shift the curve to the left (for wetter consistency), or to the right (for drier consistency), and the change in flow would be roughly proportional to the change in slump, as shown in Smith and Benham's Fig. 8, or in Lyse and Johnson's Figs. 5 to 8. That the change in slump should be more sensitive than the change in flow is also clearly indicated from the relative lengths of the ordinates through F and S and included between the curves W and D.

It has been shown in Fig. 2 that what we may call the intrinsic plasticity of a mixture (meaning that part contributed by or characteristic of the solids) is indicated by the steepness of the flow-pressure curve. Hence if we have different concretes

varying from lean to rich, or from harsh to plastic, and gage them to the same flow on the flow table, their flow-pressure curves will be somewhat as represented in Fig. 3. The point F represents the

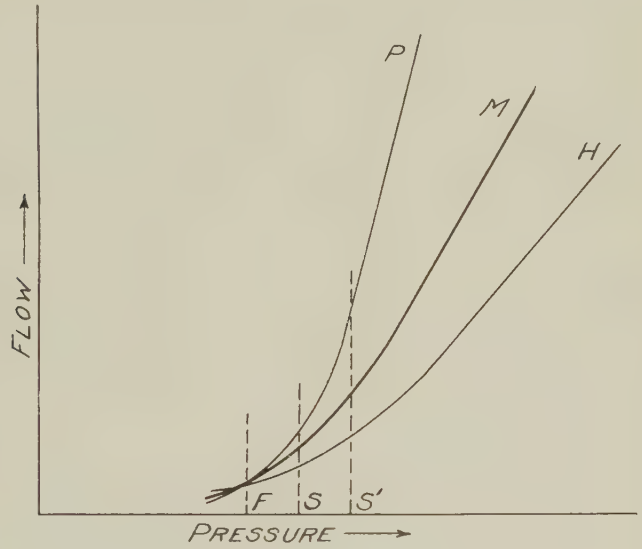


FIG. 3

force applied in the flow table test, at which point all have the same flow; the point S represents the force applied in the slump test, and it is evident at once that the plastic concretes should slump more than harsh concretes, just as the tests reported in these papers have shown.

The point of all this is that no single measure of "consistency" can possibly tell the whole story, as W. H. Herschel has so frequently insisted in his A. S. T. M. discussions[1] of papers on this general subject. If the slump test were not so crude and so lacking in dependable reproductibility, we could, as Lyse and Johnson have so ably pointed out, get some notion of relative plasticity by careful comparisons of flow-table and slump tests. But the slump test greatly needs improvement to bring about reasonable reproducibility. A complete mechanization of the

[1]See A. S. T. M. Proceedings, Vol. 25, Pt. II, p. 216, 1925; Vol. 28, Pt. II, p. 522, 1928; Vol. 29, Pt. II, p. 871, 1929.

test might help considerably, and we should be in a still better position if we could devise a more effective device, such for example as would yield the higher flows corresponding to the abscissa S′ in Fig. 3. Obviously the more points that can be obtained in these flow-pressure curves, the more certain we shall be of the characteristics of the mixtures studied.

This discussion is not intended to be discouraging to those who are interested in the practical problem of consistency control. A moderately improved slump test or a simplified flow test will serve this purpose very nicely. But for those who are interested in finding out how the workability of different mixtures can be compared and improved economically without sacrifice of other essential qualities, and what part admixtures and aggregate gradings and types contribute to this property, no single test will suffice. The ideal scheme would be to have some device that would furnish a number of well-defined points on these flow-pressure curves, but up-to-date we have found no satisfactory scheme of handling a batch of concrete in such way as to measure both force and rate of flow. Failing in this we have attempted to improve on the slump test, in the hope of using the improved test in connection with the flow-table, and thus getting two fairly definite points on the curves. But to date we have failed to overcome an undesirable lack of reproducibility, which is one of our main objections to the slump test.

The more we ponder upon this problem, the more likely it seems to us that some comparatively simple device will be developed, by means of which two or more definite applications of force can be applied to a mass of concrete, and the corresponding rates of flow measured, at least in relative terms. Perhaps we will make more progress if we deliberately turn our backs upon the present slump and flow tests and keep our thoughts focused on those fundamental relations that we want to bring out.

BY G. A. SMITH*

First I wish to consider the subject of workability of concrete in a general way. I do wish, however, to point out that, at the time our paper was presented, the sole purpose was to show that the slump test and the flow test, both used to measure the same

*Convention presentation by George A. Conahey.

property or condition of concrete, namely wetness, do not function to give comparable results. Since there was a lack of correlation in results as the mix was varied, there is conclusive evidence to the effect that other properties of the concrete were having some influence on the values obtained. I hope to take up this phase of the subject later in a separate discussion of the paper by Messrs. Lyse and Johnson.

Prof. Williams has brought out a very interesting point in the consideration of workability. Though I agree with him that segregation plays a large part when studying the property, workability, I believe that it is only one factor influencing the placeability of concrete.

First, what do we mean by workability? I am of the opinion that this involves much more than a simple consideration of the condition of the fresh concrete. In order that concrete may be of service it must be placed in the forms. For the service rendered to be of the highest quality, it is necessary that the ingredients of the concrete be uniformly distributed throughout the member. Were it possible to place concrete without effort so that a uniform and homogeneous member would result there would be no need to consider the question of workability or let us say placeability. The fact remains that concrete cannot be placed to give a satisfactory finished structure without the expenditure of effort. When I say satisfactory finished product I do not mean one which an unscrupulous artisan would pass, but one that comes as near being perfect as it is possible to make it. Uniform distribution of ingredients, complete filling of all intricate parts of the mold, and freedom from open porous structure, are conditions that must be fulfilled in the satisfactory product.

Since effort must be spent in placing concrete, be it supplied by man power, or otherwise, it is an item of cost and must be given its proportion of consideration. The amount of effort required is variable depending on the character of the concrete and the condition under which it is being placed. For example, rich mixes are recognized as being more workable than lean mixes. Concrete suitable for reinforced members is usually too wet for constructing roads, and 6-in. cobbles which are entirely satisfactory in massive structures would result in anything but a workable concrete where intricate steel fabrication and relatively

small members are concerned. Suitability for the purpose intended makes the question of workability a relative one, as Prof. Williams points out but there is ever present the fact that under all conditions work must be done to effect satisfactory placement.

Abstractly any mass of material may be workable but one may be more workable than another depending upon the amount of effort or work necessary to accomplish a definite end. In the case of concrete any mass of concrete can be worked or placed but the amount of work necessary to effect a given condition in the final product differentiates between two mixes as to their relative workability. The mass that can be placed with the least effort will be the most workable.

When placing concrete it is necessary to move small parts of the mass to remove entrained air and effect a satisfactory placement. This presupposes a non-segregating mix. In the case of a segregating mix though the concrete may fill the form fairly readily and without material effort there enters the question of distribution of the materials. If segregation has occurred, it means that a large amount of effort must be expended to bring about a redistribution of the ingredients if the satisfactory structure is to result. The remixing in the forms may be so arduous a task that it is not accomplished and the result is an inferior product. The additional work required in the case of a segregating mix may be much more than that in a mix apparently less easily worked but relatively non-segregating.

To my mind the term workability involves the question of the amount of work necessary in placing the concrete to accomplish a satisfactory finished product. On this basis segregation destroys the homogeneity of the mix and necessarily contributes to a non-workable condition requiring more work to bring about the condition we desire. The workability of a concrete, therefore, is the resultant of two factors: (1) The ease with which the aggregate particles and small parts of the concrete mass may be moved relative to each other, and (2) Cohesion within the mass or lack of segregation.

The ease, under a given condition of placement, with which movements within the mass are accomplished is dependent on the size, shape and surface characteristics of the aggregate particles and the quantity, quality and characteristics of the medium in

which they are distributed. Cohesion or lack of segregation is dependent on the same factors as those influencing the ease of movement but they do not always function in the same direction to affect the amount of work necessary to place the concrete. For example, increasing the quantity of cement, consistency and other conditions remaining constant, increases both the ease of movement and cohesion within the mass. On the other hand increasing the quantity of mixing water increases the ease of movement under a condition of uniform distribution but decreases the cohesion.

There is no need to enumerate the various items incident to the manufacture of concrete that influence workability. Each and every condition has its effect. Most of these have been discussed before this Institute and any further discussion would be superfluous. The question of admixtures and their functioning, appears to be a major consideration for this discussion. This phase I wish to discuss briefly, not with the view of comparing various admixtures but to point out what an admixture may do to improve conditions.

In any design certain conditions are laid down and under these it is expected that the requirements for the job will be met. Any addition of material to the designed mix which will influence the placeability or workability in a beneficial way may be deemed an admixture. This may be additional cement, increased quantity of mixing water, a combination of the two or some other material of merit. Should any change be made by such additions there are two conditions that must be met: (1) the quality of the resulting concrete must be unimpaired. (2) The workability of the concrete should be so improved that the placing may be expedited and result in a superior finished product.

It is generally conceded that added cement has a beneficial effect on the workability and we, not infrequently hear of an additional sack of cement being used because of its effect. The answer to this is not difficult. Since cement in reasonable amounts is necessary for a moderate degree of workability, increasing the cement paste in quantity and quality, consistency or wetness remaining constant, the ease of displacement and the cohesion in the mass are increased.

Water used to increase workability is a most unreliable admixture. Though a certain amount is necessary, depending on the job conditions, any quantity in excess of this amount is injurious to the finished product and may go so far as to cause a negative result so far as workability is concerned. Displacement in a plastic mix may be more difficult than in a wet mix but lack of cohesion in the wet mix results in segregation and additional work to effect satisfactory placement.

Other materials to be added to a mix, normally thought of as admixtures, should function to increase the ease of placement and to improve the condition of the finished product. Some materials are more effective to this end than are others just as one cement may result in a more workable concrete than another. It was pointed out that both ease of movement within the mass and segregation were dependent on the quantity, quality and characteristics of the cement paste and characteristics of the aggregates. Since the aggregates are inert, admixtures would not have any effect on them as such. The only phase they can influence is that of the cement paste.

The addition of any finely divided material and the addition of the necessary water to maintain a constant consistency tend to increase the volume of the paste. This increase in the volume of paste permits a wider separation of the aggregate particles and a consequent reduction in the resistance to internal movement. Plastic properties of the paste and concrete mass as a whole may be improved depending on the characteristics of the admixture used. Reference has been made to the fat, smooth condition noticed in concrete containing some admixtures. This condition, since it parallels that observed when the richness of a mix is increased, not only increases the ease of internal movement but also decreases segregation by increasing the cohesion within the mass.

The advantages to be derived from improving workability though not always tangible are nevertheless real. Expediting the placement and overcoming unsightly honey combing which requires patching are tangible enough in themselves but the effect of non-uniform distribution of materials which affects the durability and stability of a structure are not always apparent until it is too late to remedy matters.

It is hoped that this discussion may be helpful in bringing about a unification of ideas relative to the property, workability, so that before long when the term is used we may each have a more clear cut idea of what is meant.

The fact that the value of workability is appreciated leads me to the belief that, eventually, it will be analyzed and that a full and complete understanding will be reached.

Someone may ask the question; How is the property to be evaluated? To evaluate the property and put it on a mathematical basis would require some means for measuring the work necessary to effect satisfactory placement. Numerous attempts have been made toward this end and, if they have shown nothing more, they indicate that under comparable conditions concretes of different compositions have different properties so far as the placeability is concerned. This is an every day observation in the laboratory. When casting specimens it is observed that more effort is required to rod one concrete than is required for another. This is brought about by variation in the distribution and the resistance of the mass to the penetration of the rod. Though differentiation between mixes when rodding is at times an easy matter, it is only a relative condition that is noted and the measurement is purely qualitative. I believe that an evaluation of the rodding effort would come as near being a measure of the property as anything that could be devised. In fact this is the basic idea back of the penetration apparatus. I have used this apparatus in studying the effects of numerous admixtures and the effects of varying the proportions, grading, consistency, etc., and have obtained indices for the workability which appear to parallel interpretation as to the relative workability of the concretes considered.

In conclusion I wish to say that, with the extensive thought and consideration now being given to this phase of concrete it is my firm conviction we will not have to wait long for an answer to some of our problems. We will have in the near future a definite correlation of ideas on the subject and an accepted method of evaluating separately, if not collectively, the two factors governing workability namely, ease of movement in the mass and segregation.

DISCUSSION OF LYSE AND JOHNSON PAPER BY G. A. SMITH

The editorial comment introducing the two papers, one by Messrs. Lyse and Johnson on the subject of the consistency of concrete and the other by Mr. Benham and the writer, creates an erroneous impression relative to the attitude of the several authors. The comment indicates that the writer favors the flow-table as a yardstick for measuring workability whereas Messrs. Lyse and Johnson favor the slump test. I wish to point out that in no instance in the paper "A Study of the Flow-Table and the Slump Test" was the flow-table indicated as a measure of workability. The authors certainly did not at any time have in mind indicating that either test is a measure of workability when concretes of different compositions are considered. When referring to the property or condition that was being measured the terms consistency and wetness were used.

Though the workability of *a given mix* will vary, within limits, with the quantity of water used or with the wetness as indicated by either the slump or the flow, the workabilities of two different mixes gaged to either the same slump or to the same flow may be entirely different. The point is, that workability and wetness (consistency) are not synonymous under all conditions. In this case the use of the two conditions or properties interchangeably leads to a misunderstanding and a misrepresentation of the authors' conclusions. I quote from the last paragraph of the conclusions: "The data on the whole indicate that, where a consistent indication of the consistency or wetness of concrete is desired, the flow test is preferable to the slump test because of its more nearly uniform functioning." The only condition indicated is *consistency* or wetness.

The Editor in introducing the two papers asks, who can discover a common ground. It appears to me that both papers have very much in common and when properly interpreted will indicate the same conditions and that the same conclusions may be reached from either paper.

It was with a great deal of pleasure and interest that I reviewed the paper "A Study of the Slump and Flow of Concrete." Particularly because Messrs. Lyse and Johnson showed tendencies of results where conditions were comparable that paralleled those

observed from our tests. No comment is necessary in these instances. On the other hand there are one or two conditions on which I wish to comment.

The authors indicate that the use of the slump specimen in determining the flow results in larger flows and consequently is a more sensitive measure than is the flow obtained with the usual flow mold. This is quite obvious but it has little to do with the apparatus or the condition of the concrete. It is largely a question of mathematics. Consider the conditions existing in the test. The volume of concrete in the flow specimen is about 0.16 cu. ft. and the basic diameter is ten inches. In the slump specimen there is approximately 0.20 cu. ft. of concrete and the basic diameter is eight inches. Other conditions being equal it is seen that the flow increase in diameter with the slump specimen should be about 1.6 times as great as that using the flow specimen. There is, however, the effect of the additional head and that of the erratic functioning of the slump test which tends to give a greater spread. The increased head due to the height of the slump specimen does not appear to have a large effect since the ratio of the increase in flow for the slump specimen to that for the flow specimen between the zero and the five drops (Figs. 1, 2 and 3) varies from less than 1.0 to 1.7 averaging less than 1.3. Were the functioning consistent the increase in flow would be about 1.6. The functioning of the slump test has been discussed in the paper by Mr. Benham and the writer. It was pointed out that in the wetter mixes the concrete flowed down and out and the test was not a true slump. In the tests by Lyse and Johnson this same condition undoubtedly existed as is evidenced by the high flows at zero drops and was unquestionably responsible for at least a part of the increased flow observed. Personally, the writer is inclined to favor flows of lower magnitude because of the more nearly rectilinear relation that exists between flow and the quantity of water used. After the mass of concrete has spread a certain amount this relation no longer exists but becomes a curvilinear relation since the area covered by the sample varies as the square of the diameter. No specific example can be cited but it is readily seen that during the early flow there is a reserve supply of sample which is apparently used up in compensating

for the increased area. After this reserve has been used and the concrete has spread to nearly uniform thickness proportionate changes in diameter become less for increased manipulation and for additional quantity of mixing water. This is evidenced by the diminished rate of flow with increased number of drops for the wetter consistencies, (See Figs. 1, 2 and 3 of Lyse and Johnson paper) as well as lowered increased in flow for a given increase in water.

Lyse and Johnson make numerous references throughout the discussion and in their conclusions to the effect that the slump test is superior to the flow test for measuring workability. This is based on the observation that for a given flow the slump for the richer and more workable mixes was greater than that for the less workable mixes. The variation in slump with observed improvement in workability, the flow being constant, was observed and pointed out by the writer.

It is believed that the difference in slump, flow or consistency or wetness being constant does parallel improvement in workability. However, to say that, because of this, the slump is superior to the flow as a measure of workability is far from being logical. Let us consider the converse of this condition of equal flows and variable slumps. For equal slumps and flow would be less for the more workable mixes. It would be, then, just as logical to say that the flow was a more sensitive measure of workability, the workability varying inversely as the difference in flow.

Neither test in itself can be used as a criterion. Supposing that either the slumps or the flows were constant for two mixes, one lean and the other rich. The fact that the slumps were constant according to the authors would indicate that one was equally as workable as the other. This we know is not the case. Each of the methods is a measure of the wetness or consistency of the concrete and individually for a given mix is a measure of the relative fluidity of the mix. The only correlation between slump or flow and workability is in the case of a given mix. With a variation in quantity of water both the slump and the flow vary and so does the workability within limits. Mixes varying in proportions with variable materials cannot be correlated as to

relative workability by either of the test methods used independently.

In conclusion I believe that the Editor's question has been answered and that we find far more in common in the two papers than there is in disagreement. There is one differentiation that must be made relative to two properties of concrete. Under different conditions of composition and under variable conditions as relate to the properties of the separate ingredients of concrete, it is absolutely necessary to distinguish between fluidity, wetness, consistency or whatever we choose to call it and the workability. Once this is done we will be in a position to study the effects various materials and variations in materials have on the workability or the placeability of concrete.

BY DONALD M. BURMISTER*

These two papers are valuable contributions to the study of concrete mixtures and contain very complete data showing the variation of slump and flow resulting from changes in cement and water content in various concrete mixes. The interpretation of the data by the writer leads to the conclusion that these tests measure different properties of the concrete mixtures, both of which, however, must be included in any criteria for workability. In the following discussion the writer presents a study of these data and suggests a means of combining the test results in order to arrive at a measure involving both properties of concrete.

The handling and placing properties of concrete may be expressed in two ways, first on the basis of the effort and time to produce the finished product, and second on the basis of observed effects of certain characteristics by which workability may be judged. The properties of concrete are reflected in varying degrees in its behavior in every test made upon it as well as upon the job. Its more or less fluid nature imparts flow characteristics. Its semi-plastic nature imparts smoothness and mobility. These two qualities together determine to a large extent the workability of concrete. These qualities as criteria of workability in concrete may be used as bases of design of concrete mixtures or of the comparison of concretes.

In order to evaluate the slump and flow tests, it is necessary to determine how these two tests function and which qualities of

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a concrete mix have the greater relative effect, for which that test then may be taken as a relative measure.

The flow table—Primarily the flow table brings out the fluidity of the concrete due to wetness. Flow is a property of a fluid condition. A more or less true flow is produced by the work performed upon the concrete when the mass has been broken down or has slumped down, so that a quite uniform slope exists radially outward from the center.

This is indicated in Fig. 1, 2, and 3, by Messrs. Lyse and Johnson, who have plotted in the lower right hand quarter of each figure the effect of the number of ½-in. drops of the flow table upon the flow, using the flow cone. The effect of some slumping action in increasing the flow from zero to five drops is shown in nearly every case by the greater steepness of the curves in this range. Thereafter the curves assume an almost uniform slope, showing that the flows for successive drops were very nearly equal, and were proportional to the work performed upon the concrete. Therefore a better measure of wetness is obtained by the larger number of drops, for instance ten or fifteen.

It seems to the writer that a better and more sensitive measure of wetness is obtained by using that half of the circumference of the concrete mass, on which the greatest flow occurs as the average figure. The lopsided flow, if the flow cone has been properly centered, is an exaggerated condition, due mainly to non-uniformity of depositing the concrete. But this condition may reflect a harshness and a tendency toward segregation in the concrete. The uniform slope of the curve mentioned above and the almost regular increase in flow with increase in wetness shows that the flow table measures the wetness of concrete.

The slump cone—The slump cone measures primarily the plasticity of a concrete, which is imparted to the mass by the fine material in the mortar, whether it be cement, silt and clay, or other admixtures. The differences in the properties and the behavior of workable concretes and those classified as non-workable are largely due to this fine material, which acts as a lubricant and more important still tends to retain all of the mixing water within the mass. Consequently smoothness and mobility are imparted to the concrete, which varies with the amount and the kind of this fine material and also with the amount of mixing water.

In plastic concrete the particles of coarse aggregate, when separated by sufficient plastic mortar, flow more smoothly over one another than in a non-plastic harsh one, because of the low shearing resistance. Consequently the mass as a whole deforms more readily. Therefore under the action of gravity the plastic concrete will slump more for the same degree of wetness.

The slump cone gives a real measure of this quality plasticity only when a true slump is obtained as indicated in types 1 and 2 by Messrs. Smith and Benham. If type 4, in which the concrete slushes down leaving a compacted core, indicates anything, it shows that there has been a breakdown in the binding properties of the mortar, due to an excess of mixing water in relation to the given mix. There will therefore be a greater tendency for segregation and for the leaking of the liquid from the concrete.

In these two tests two qualities have been relatively measured, or rather the effects of these two qualities:

1. The flow table reflects the wetness of the concrete.

2. The slump test reflects the plasticity of the concrete. Of course in both tests both qualities have some effect, but the nature of the tests are such as to bring out the particular quality more effectually.

A method of relatively evaluating the effects of these characteristics of concrete is outlined, resulting in the plasticity coefficient, which is suggested by the writer as an index or yardstick of relative workability. For any given concrete the plasticity coefficient is the ratio of the slump in inches to the inches of radial flow on the flow table.

The Plasticity Coefficient equals $\frac{\text{Slump in inches}}{\text{Radial flow in inches}}$. It reflects the relative plasticity and wetness of the concrete and is indicative of the behavior of concrete during handling and placing. The purpose of such an index is to provide a figure which will be indicative of the characteristics, and one which may be used as a basis of comparison and design of concretes, which the slump or flow alone have not effectually done. The properties and behavior of any concrete are made more tangible and the concrete is more completely defined by:

1. The plasticity coefficient and the flow. or

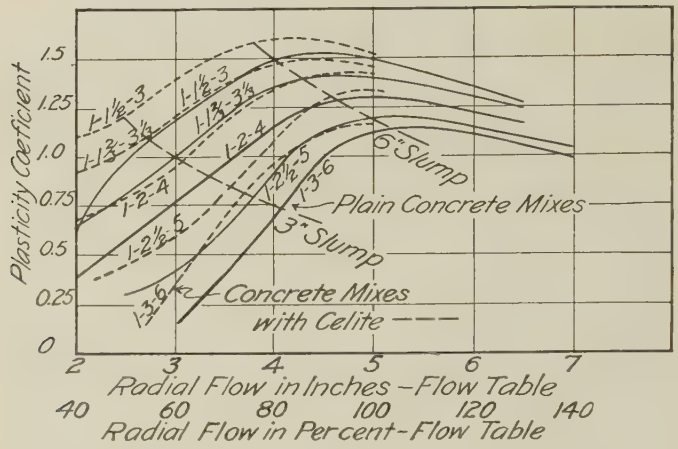


FIG. 1—RELATIONS OF PLASTICITY COEFFICIENT, FLOW AND MIX

2. The plasticity coefficient and the slump.

When the curves in Fig. 8, "Relation between Flow and Slump" by Messrs. Smith and Benham are replotted with the plasticity coefficient as ordinates and the corresponding flows for the different mixes as abscissas, a most interesting set of curves results. The flow is chosen rather than the slump because the

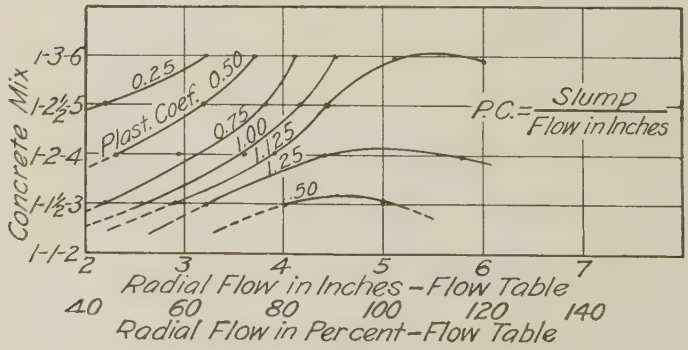


FIG. 2—THE VARIATION OF THE PLASTICITY COEFFICIENT WITH FLOW FOR PLAIN CONCRETE

range is greater and the curves bring out certain facts better. A flow of 100 per cent on the flow cone of ten inches in diameter, for instance, corresponds to a radial outward flow of five inches, and others in proportion. The data taken from these curves is tabulated in Tables 1 and 2.

These results have been plotted in Fig. 1, showing the relation of the plasticity coefficient, flow and mix. They have been replotted from the curves of Fig. 1 in a different form in Fig. 2, which brings out better the relations existing between these factors.

The following conclusions are obtained from these curves.

1. The plasticity coefficient increases with the richness of the concrete. A high plasticity coefficient, as a criterion of the desirable qualities of concrete, is indicative of a uniform and homogeneous product with little tendency to segregate.

2. The plasticity coefficient also increases with the addition of water up to some maximum point, which is the limit of workability for that concrete. However, this maximum point probably results from the fact that the slump test ceases to function as such for slumps greater than seven inches.

3. The addition of Celite as an admixture increases the plasticity coefficient, and that for a given flow concrete with Celite admixture has the equivalent plasticity coefficient of a much richer mix, as indicated by the relative positions of the curves, thus indicating an increase of workability.

4. The two figures plasticity coefficient and flow or slump are quite definitely indicative of the relative ease with which a concrete may be handled and placed, and are therefore bases for the comparison of concretes, as to their handling and placing properties.

5. A low plasticity coefficient either indicates lean watery concrete which is harsh and will tend to segregate badly, or it indicates a very dry harsh mix which is difficult to place. Any considerable working to make such concretes flow into form work will only aggravate its undesirable tendencies.

6. In Fig. 1 two lines have been plotted showing the relation of slumps of 3 and 6 in. to both the plasticity coefficient and flow. These show that for equal slumps the leaner mixes are much wetter and more fluid than the richer mixes as indicated by their

respective flows, and also the plasticity coefficients are lower for the leaner mixes. Considering equal flows, the richer mixes have greater slumps and the greater plasticity coefficients, and there-

DATA TAKEN FROM FIG. 8, "RELATION BETWEEN FLOW AND SLUMP" BY GEORGE A. SMITH AND SANFORD W. BENHAM

TABLE 1

Flow		Plasticity Coefficients for Plain Concretes									
Per Cent	Radially in Inches	1:3:6 Slump	1:3:6 Pl. C.	1:2½:5 Slump	1:2½:5 Pl. C.	1:2:4 Slump	1:2:4 Pl. C.	1:1⅔:3⅓ Slump	1:1⅔:3⅓ Pl. C.	1:1½:3 Slump	1:1½:3 Pl. C.
40	2.0					.8	.40	1.3	.65	1.2	.60
50	2.5			.8	.30	1.4	.56	2.0	.80	2.5	1.00
60	3.0	.5	.17	1.3	.43	2.3	.77	3.0	1.00	3.5	1.17
70	3.5	1.4	.40	2.2	.63	3.4	.97	4.2	1.20	4.8	1.33
80	4.0	2.7	.68	3.7	.93	4.7	1.17	5.4	1.35	6.0	1.50
90	4.5	4.6	1.02	5.2	1.15	5.7	1.26	6.3	1.40	6.8	1.50
100	5.0	5.6	1.12	5.9	1.18	6.4	1.28	7.0	1.40	7.4	1.47
110	5.5	6.2	1.13	6.5	1.18	7.0	1.28	7.4	1.35	7.9	1.43
120	6.0	6.6	1.11	6.9	1.15	7.4	1.23	7.8	1.30	8.3	1.39
130	6.5	6.7	1.03	7.2	1.11	7.7	1.19	8.0	1.25	8.4	1.19
140	7.0	7.0	1.00	7.3	1.05	7.8	1.11				

TABLE 2

Flow		Plasticity Coefficients for Concretes with Celite Admixture									
Per Cent	Radially in Inches	1:3:6 Slump	1:3:6 Pl. C.	1:2½:5 Slump	1:2½:5 Pl. C.	1:2:4 Slump	1:2:4 Pl. C.	1:1⅔:3⅓ Slump	1:1⅔:3⅓ Pl. C.	1:1½:3 Slump	1:1½:3 Pl. C.
40	2.0					1.3	.65	1.8	.90	2.2	1.10
50	2.5			1.2	.48	2.0	.80	2.7	1.08	3.0	1.20
60	3.0	1.0	.33	1.7	.57	2.8	.93	3.6	1.20	4.3	1.40
70	3.5	2.3	.66	2.7	.77	4.1	1.17	4.8	1.37	5.4	1.55
80	4.0	3.8	.95	4.4	1.10	5.3	1.33	5.9	1.48	6.4	1.60
90	4.5	5.2	1.13	5.8	1.31	6.3	1.40	6.7	1.48	7.2	1.60
100	5.0	6.0	1.20	6.5	1.30	7.0	1.40	7.3	1.46	7.7	1.54
110	5.5	6.5	1.18	7.0	1.27	7.4	1.31	7.8	1.42	8.2	1.49
120	6.0	6.8	1.15	7.3	1.21	7.7	1.28	8.1	1.32		
130	6.5	7.0	1.08	7.4	1.15	7.8	1.20	8.3	1.28		
140	7.0	7.2	1.03	7.7	1.10	8.0	1.14	8.4	1.20		

Note—Pl. C. is the Plasticity Coefficient

fore possess more of the desirable qualities by which workability is judged.

7. Similar curves for other materials will reflect the effect of grading and proportions upon the workability of the concrete made from them.

Basis of comparison—In classifying a concrete it may be more plastic and less wet or it may be wetter and less plastic. Any real basis of comparison must rest upon measures of both of these characteristics in concrete. The two concretes are quite different in their properties and behavior and are judged accordingly as to their relative workabilities. From the foregoing discussion it is seen that there are two bases of comparison of concretes, that are quite definite and simple of application.

1. The comparison of plasticity coefficients for equal flows.

2. The comparison of plasticity coefficients for equal slumps. The ones with the greater plasticity coefficients have more of the desirable qualities by which workability in concrete is judged.

Basis of design—From the design standpoint there are two simple requirements which can be specified in addition to the water-cement ratio governing strength.

1. A minimum value for the plasticity coefficient consistent with the class of concrete and the field conditions.

2. A maximum flow or slump suitable for the given field conditions.

In meeting these requirements a concrete of pretty definite characteristics and uniformity is obtained. In this connection the curves in Fig. 2 are very useful, or similar ones. For example, if a flow of 70 per cent or 3½ inches on the flow table was desirable and suitable for the given field conditions and further if a plasticity coefficient of at least 1.00 was specified, the leanest mix that could be used for the given materials and proportions of aggregates would be a 1:2:4 plain concrete. The necessary slump that must be obtained is readily found from the relation—
Slump equals Pl. C x Flow in inches.

This simply means that the concrete has the desired properties and workability for the given job, when the flow is 3½ in. and when the slump is also 3½ in. at the same time. To meet both criteria at the same time in a concrete mix is to ensure the quality specified, whether obtained by the use of more cement,

admixtures, or better by such gradings and proportions of the aggregates as are necessary. This may be accomplished by trial methods, starting with a cement paste of given water-cement ratio, as specified for strength. When the mix has been properly designed, using both tests to fix its characteristics, then the proper field control may be maintained by means of the slump test alone, as an indication of the uniformity of the output.

Thus by combining the results of the flow table and the slump tests, the properties and behavior of any concrete are made more tangible and the criteria of plasticity coefficient and flow or slump serves as a more definite basis for designing concrete mixtures and for the comparison of different concretes.

BY F. O. ANDEREGG

The method of attack on the workability problem proposed by Mr. Pearson of using the pressure-flow rate curve is, I believe, the best that has so far been suggested. After seeing these curves it is obvious why the slump result may not be consistent with the flow table finding. It should be pointed out, however, that the flow rate is usually linear while both slump and flow table measure flow in two dimensions.

The best definition of workability involves both ease of working the concrete into place and freedom from segregation. The latter is a result of improper grading somewhere in the system: The water may flow from the cement if the latter is not properly ground; the cement paste may have a tendency toward oozing out of the sand if the latter is improperly graded; and finally, the mortar and coarse aggregate may separate from each other.

By building up a system with a proper gradation of sizes we have in the first place a sort of filter which prevents segregation. It is a little like a sand filter except that here we want a system where large particles serve as a sort of anchor or key holding back the next smaller size and so on down to the finest cement particles. In turn all the smaller sizes act as ball bearings permitting the ready rolling of the next larger into place.

The result is a maximum of ease of placing, a minimum of segregation, together with a maximum of packing and very good workability is secured with a minimum of lubricating water.

To secure this ideal grading use a set of Tyler standard screens differing in opening dimension by the factors and the ratio of fractions on the separate sieves is constant at about 1.2, the increase in weight (or absolute volume) being greater as the sizes increase. This ratio is affected slightly by the shape of the particles.

On a basis of the ideal grading we can go ahead logically with the analysis of the workability of commercial concretes. The latter differ from the ideal by gaps in the grading curve, or humps where too great a preponderance of one size occurs, or in combinations of the two. The procedure would then be to determine complete pressure-flow rate curves at several degrees of wetness for ideally graded materials and then introduce systematically humps and gaps into the curve, and determine the effect. When sufficient experience has been accumulated, one should be able to predict accurately what any given commercial grading will do and give it a grade, say 50 per cent or 70 per cent of the ideal.

On a basis of an ideal grading it is not difficult to secure an understanding of the role of *admixtures,* other than specific waterproofing materials or hardeners. If the cement is not ground to approximate the ideal (and few cements are in the very fine region) a finely ground, well-graded admixture may aid in filling voids and serving as ball-bearings. In this way a real improvement may be made.

Again, if the aggregates are quite harsh it is necessary to provide a greater bulk of paste to prevent segregation and containing more lubricating water. With bentonite clays and diatomaceous earths we can get more lubricating water into the system. At the same time the water-cement ratio is increased and whether the addition effects an all-around improvement depends upon the grading curve for the whole system, cement-sand-coarse aggregate. With properly ground cement and properly graded aggregate an admixture would be not only superfluous, but actually harmful.

FURTHER DATA WITH DISCUSSION FROM G. M. WILLIAMS

The writer has not at this writing seen the discussion relating to his paper "Admixtures and Workability of Concrete" but some of the criticisms and questions may be partially answered

by the following data obtained in tests with a type of admixture which was not represented in the original tests.

The writer's definition of workability and the effect of admixtures is summarized as: By workability is meant the ability of a concrete mixture to remain homogeneous and free from segregation during the process of transportation and placement. A non-segregating mix, having necessary flowability, is a workable mix. Segregation is influenced by quality and quantity of mortar in the mix and occurs when mortar is so fluid that it cannot adhere to the large aggregate particles and hold them in suspension, or when the relative mortar content is too small. Workability increases and segregation decreases as volume of mortar increases which results in a heavier, stickier, better adhering mortar without loss of necessary flowability. Various powdered admixtures including portland cement, were found effective in increasing relative mortar volume but their efficiencies varied greatly. The efficiencies were approximately proportional to the volumes of pastes formed when equal weights of admixtures were brought to the same flowabilities. Identical concretes containing equal volumes of admixture pastes were found to result in practically the same segregations as measured by spread of aggregate beyond the mortar line in the flow table.

While one admixture used in the original tests contained a percentage of bentonite or colloidal clay its behaviour was not the same as has been found for Colloy and Aquagel, two colloidal clays which are representative of this type. While some work has been done with both of these materials the data following relates mainly to the use of Colloy as an admixture to increase workability.

Colloidal clays or bentonite have the property of absorbing many times their weight of water, resulting in the formation of a very large paste volume. When used as an admixture in concrete a large increase in mortar volume should be expected, due to the addition of a relatively small weight of the material in powder form. Volumes of pastes formed by the addition of water up to 300 per cent by weight are shown for Colloy, Aquagel and Celite in Fig. 1. This water quantity is just beyond the saturation limit for Celite. It will be noted that the volumes are practically the same and proportional to water content up to this limit.

Comparing Celite and Colloy identical concretes were prepared to which were added equal weights of admixtures. Flowabilities were made equal, amount of segregation noted, and mortar volumes computed as listed in Table 1.

TABLE 1—AVERAGE MORTAR VOLUMES AND AMOUNTS OF SEGREGATION FOR EQUAL WEIGHTS OF ADMIXTURES

Weight of Admixture grams	Volume Mortar c. c.	Increase in Mortar Volume due to Admixture c. c.	Added Mixing Water Required by Admixture grams	Segregation of Coarse Aggregate grams	Flowability
0	763	...	...	197	177
48 Colloy	821	58	38	148	174
48 Celite	883	120	100	41	175

As indicated in Table 1 the large volume increase in mortar content which would be expected for Colloy did not result. The foregoing figures indicate that approximately 100 grams of Colloy would be needed under these conditions of test to furnish a mortar volume resulting from the use of 48 grams of Celite.

In Table 2 are shown the results when the Colloy content was adjusted to furnish the same mortar volume.

TABLE 2—AVERAGE MORTAR VOLUMES AND AMOUNTS OF SEGREGATION ADMIXTURES PROPORTIONED FOR EQUAL MORTAR VOLUMES

Weight of Admixture grams	Mortar Volume c. c.	Segregation of Coarse Aggregate grams	Mixing Water c. c.	Flowability
100 Colloy	886	36	390	172
48 Celite	885	22	400	180

In the foregoing experiments the Colloy was added in dry powder form with the other constituents, as is the usual practice and as is recommended by the producer of this admixture. In no case, when this procedure was followed, was it possible to obtain the bulking effect for Colloy which would be expected from a study of Fig. 1. Similar results were also found for Aquagel.

However, when the admixture was pre-hydrated, by the formation of a paste prior to its addition to the volume of concrete, mortar volumes were increased in a manner which were considerably greater than could be predicted from Fig. 1. Results obtained by the two methods of adding the admixture are given in Table 3.

TABLE 3—COMPARISON OF MORTAR VOLUMES AND SEGREGATIONS OBTAINED WHEN COLLOY WAS ADDED AS (1) POWDER AND (2) A PASTE

Admixture	Total Mixing Water c. c.	Flow-ability	Volume Mortar c. c.	Segre-gation grams
48 Colloy (Powder)	365	192	840	100
48 Colloy (Paste)	565	187	1000	15

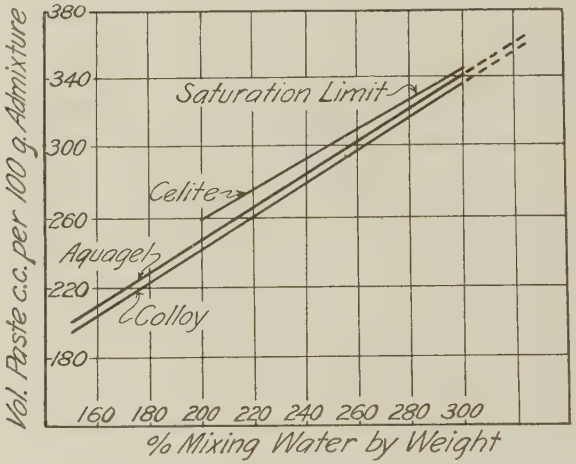


FIG. 1

The use of prehydrated Colloy resulted in the use of a greater total quantity of mixing water than was needed when the dry powder was added and in turn the mortar volume was greatly increased. The fact that some segregation occurred in spite of the large mortar volume in the batch, indicates this mortar to have lost some of its adhesiveness and power to hold the aggregate particles in suspension. The density or specific weight of this mortar must be less since water furnishes increase in volume, the solid content being the same.

The foregoing test results indicate that conclusion No. 2 of the original paper cannot be readily applied to admixtures of the bentonite type. Increase in volume with increase in water content in paste form is not a criterion of volume increase when the admixture is added to the concrete batch in the usual manner. If an admixture of the bentonite type is prehydrated before

addition to the mix its effect on mortar volume can be predicted from the volume of paste formed but this paste, due to its high water content, cannot be as efficient as one formed by the other types of admixtures, when compared volume for volume. The neat paste test will also fail in application to admixtures of the oil or tar type. Therefore it seems best to make use of the flow table segregation test directly in rating admixtures. Volume of paste formed is a matter of computation, and efficiency of the mortar in retaining and holding aggregate particles can be measured quantitatively by noting segregation. If mortars are dissimilar as may be the case in comparing a bentonite mortar with a Celite mortar, admixture quantities can be adjusted until the segregation is the same before decision as to relative efficiencies is made. In this manner the flow table segregation test can be employed to determine the efficiency of any admixture, whatever its type, and the procedure outlined above and in the original paper may be summarized as follows:

Step No. 1—Prepare a basic mixture of cement, graded sand and ¾-in. to ⅜-in. aggregate with the latter in excess so that when sufficient water is added to result in a non-fluid or segregating mortar (a plastic, adhesive mortar) there will be considerable segregation of coarse aggregate beyond the mortar line of the flow table.

Step No. 2—In addition to a basic mix plain concrete, prepare an additional batch for each admixture to be tested. Use specified quantities of admixtures and sufficient mixing water to result in the same flowability for all. Compute mortar volume of each batch and note the amount of segregation. Values obtained should be an average of at least three trials in which concordant results are obtained.

Step No. 3—Adjust admixture contents so as to furnish equal mortar volumes for each. Make flowabilities equal, compute mortar volumes and note segregations. After three concordant trials, if segregations are not approximately equal, further adjust admixture contents until equal segregations are obtained. Mortars lacking in density and adhesiveness to hold aggregate particles in the mass can be improved by the addition of more admixture in the case of powdered materials, which in turn tends to furnish a less fluid mortar of still greater volume without loss

of necessary flowability. Some adjustment of water content must be made with any change in admixture content when using materials of the powder type. In making the flow and segregation test particular care should be employed to insure uniformity in placement of concrete in the mold since those aggregate particles in the outer portion of the mass appear as segregated particles. Pockets of excess coarse material placed in the center of the mass will furnish too small segregation volumes and lead to improper conclusions.

Readers are referred to the JOURNAL *for Nov. 1931 (Vol. 28) for further discussion. All additional discussion should reach the Secretary by Oct. 1, 1931.*

Discussion of Report of Committee 108

"Properties of Mass Concrete"*

BY RAYMOND E. DAVIS†

The published report in the January Journal, is preliminary. I should like here to emphasize some of the things touched upon in the report and perhaps bring to your attention informally, certain other considerations, not previously presented. What I have to say will be more concerning what we do not know about mass concrete than what we do know. As a matter of fact, we know very little. Committee 108 was organized last spring because some of us felt the time had come to find out what is actually going on inside of concrete in large masses. The committee has done very little except to survey the field. The report presents a digest of material which has been gotten together by the committee giving available information which has been collected on various projects, principally in the field. The data have to do principally with dams—to some extent with other structures such as bridges and massive foundations. In examining the material, you will find that practically all of it has to do with the temperatures that have been observed to occur inside of these structures. We all know that cement and water, when mixed with aggregates to form concrete, evolve heat during the hardening process. In our small structures of thin sections, we do not notice that heat; but in the more massive structures, it becomes an important factor. We know something of the properties of concrete when cured at normal temperatures in small masses; but we do not know very much about the properties of concrete when cured at higher temperatures in large masses. Observations

*A. C. I. Journal, January 1931; *Proceedings*, Vol. 27, p. 385.

†Remarks by Professor Davis in formal presentation of his previously published Committee report to the 27th Annual Convention, Milwaukee, February 24-26, 1931. Following the Committee Chairman's introduction is the discussion, which will be continued in the Journal for November, 1931, V. 28.—Editor

made on various structures, even of the leaner concrete, show increases of temperature to 40 and 50 deg. above normal, or above the initial temperature of the concrete at the time of placing. With richer concrete we sometimes get increases of more than 100 deg.—in fact, in some cases, the cement paste boils. We have been taking it for granted that properties observed in concrete in small masses—properties of strength, elasticity, plasticity, durability, permeability—are to be found also in concrete of large masses, when, in point of fact, we do not know. To learn what we do not know in this regard is the job that faces our committee.

In the few tests made here and there in laboratories, notably some of the tests we have been making during the last couple of years, at the University of California, where we have specimen sizes up to one cubic yard, we have learned something of what happens to the strength under the temperature conditions that obtain inside the larger masses. We know that heat hastens the chemical process of hardening, and that at the early ages we get very much greater strength than at the normal standard temperature of 70 deg. But we know that concrete cured at the lower temperature gradually increases in strength until it passes that obtained with the higher temperatures that obtain inside the mass. In some cases there is a slight indication that there is a retrogression of strength when cured at the very high temperature. This Committee hopes to determine, both by laboratory tests and by field investigation, what is happening in our concrete structures and what are the properties of concrete cured in masses or under conditions simulating mass concrete. After concrete is placed, in a dam, for example, there is evolution of heat and the temperature rises. It continues to rise until the rate of heat evolution equals the rate of dissipation of heat, when the temperature curve, if you please, flattens out, becomes horizontal, and thereafter there is a gradual decline. In some of these structures the concrete does not come back to normal temperature until several years after placement. What are the stress conditions inside the structure? Certainly during the evolution of heat and increase in temperature there has been a tendency to expansion along the axis of the dam and then there is a compression, the concrete is then more or less in a plastic

state. It flows, if you please. When finally the peak has been reached, it is perhaps in a maximum state of compression, but thereafter, with the decline in temperature, there, a tension takes place before it gets back to normal temperature, and that may produce cracking, not only in the direction up and down stream, but perhaps along the axis of the dam itself, vertical cracks which in themselves may be very dangerous. Instead of a dam acting as a cantilever sticking up there, it might be acting as a succession of cantilevers, one leaning on the other like a pile of boards, and we might have a collapse.

Now the course of investigation would seem to be, first of all, to find a cement which would not evolve any heat, if such a thing were possible. While that is not possible, we are directed to a desirability of a cement with low heat evolution; perhaps a type of cement different from that we now employ. We have gone to quick-hardening cements, splendid for certain purposes, but generally involving sudden and large evolution of heat. Other things being equal, we would expect the finely ground cement showing more heat evolution. Chemical compositions generally found in our quick hardening portland cements would make for large heat evolutions. Perhaps not even our normal cement which we have been using for years past, is the best cement for dams or other mass concrete construction. We may come to use a cement of relatively low heat evolution—perhaps of coarse grains or high silica content, or the cement manufacturing process may play some part which will give us a low heat evolution—a cement which will gather its strength slowly, the concrete remaining in a semi-plastic condition for a longer time and therefore affording opportunity for adjustment within the mass, as it expands and contracts, so that finally there will not be left within the mass any large residual stresses. That would be a desirable type of cement for mass concrete construction. Instead of having two types of cement, the normal portland cement and quick hardening portland cement, we might have still a third for mass concrete construction—a slow-hardening portland cement of low heat generation. This committee comprises 25 members—a representative group of engineers of considerable prominence in the fields of mass concrete construction and of research. Many

of its members are in our western territory, where they are building most of the large dams, men who have been engaged in dam design and construction, some of them for a good many years, together with some of our eastern men, who are experts in the field of research. The work has been divided among four subcommittees—Laboratory program, Field program, Cement and Admixtures and Executive. The committee has had two meetings at which the cooperation of groups representing projects of large magnitude has been pledged—California's Department of Dams, the U. S. Bureau of Reclamation, the Los Angeles County Flood Control District, and others; and likewise we have the promised cooperation of the manufacturers of cement and admixtures, not only in furnishing materials and personnel for carrying out these tests under centralized administration, but also in the way of actual monetary contributions which are important, gentlemen, in tests of considerable magnitude.

It is all right in our ordinary tests to play with little briquettes, two-by-four or six-by-twelve cylinders, but when you get into mass concrete, gentlemen, it becomes a different story. There you have not only to make observations in the field under field conditions in the actual structures, but you have got to make in your laboratory, specimens of unprecedented size. In the program of tests we have laid out, some of our smallest specimens are cubic yard masses; some of our large specimens contain fifty cubic yards. One test proposed is upon cylinders 12 ft. in diameter by 12 ft. high, in each of them containing approximately 50 cu. yds of concrete. In carrying out this particular test we have not only to manufacture the cylinders but to seal them up in such a manner that they will neither give up nor absorb moisture, and keep them in a special room where the temperature will be constantly maintained outside at the same temperature as that inside the mass and automatically regulated. Inside each of these 12-ft. cylinders, devices will be imbedded for measuring the strains of the mass. Smaller cylinders will be embedded in the large and their properties determined.

We are desirous of securing the cooperation of all those who have anything to do with mass concrete construction, who have not already come forward and volunteered their services.

CONVENTION DISCUSSION

CHAIRMAN HOLLISTER—The subject of mass concrete is assuming at this time particular importance with the impending contracts for the construction of the Hoover Dam. Normally we think of 6,000 yards of concrete as being a fair sized job. One unit of the Hoover Dam itself will take 6,000 yards a day for two years, about three and a half million cubic yards of concrete in the mass of the dam itself, not including minor appurtenances running to over a million yards. The chairman of the committee on Properties of Mass Concrete is a member of the Board of Consulting Engineers on concrete on this dam. The Board consists of five men under the chairmanship of Mr. Bates, all five of them are members of this Institute.

L. G. LENHARDT—Talking about millions of yards of concrete and 6,000 yards a day for two years rather startles me. I have a few data here that I happened to jot down. On a tunnel job—12 ft. diameter and some hundred feet beneath the surface, ground temperature 55 deg.; mix used, approximately 1:2:3; water-cement ratio varied between five and a half and six and a half gallons; temperature of concrete on placement, 65 to 70 deg.; wall thickness 21 in.; temperature of the concrete, four different readings, 118 deg., 122 deg., 123 deg., 125 deg.; maximum at three days after placement. The same temperature held four days and showed slight signs of dropping off at five days. The concrete was placed at the rate of 8 to 10 yds. per hour. On another job, caisson, 8 ft. walls, outside temperature, 18 to 20 deg., concrete placed at 45 deg.; placed at the rate of 32 to 40 cu. yds. per hour with a total of 2,600 cu. yds. in the pour. Pried back face of forms at the end of 24 hours and found a temperature of 80 deg. at the face of the concrete. We had a pipe insert extending four or five inches back in the concrete and the temperature at that point was about 85 deg.

WRITTEN DISCUSSION

BY CLIFFORD BETTS

The summary of existing data on "Properties of Mass Concrete" by Committee 108 logically deals primarily with the chemical generation of heat and resulting expansion and contraction of the mass. This phase of mass concrete behavior merits intensive

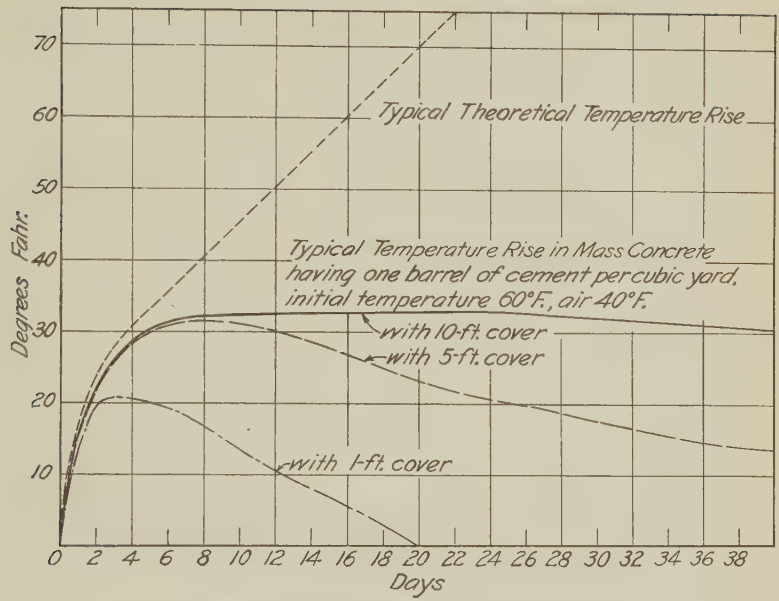
study in both the field and laboratory in order that design of large structures may not be handicapped by inadequate data. The committee has performed a distinct service in presenting a summary of research to date and a proposed program of investigations whereby future investigations may be coordinated.

Shrinkage stresses in massive concrete structures—notably dams, piers and arches—may even exceed loading stresses unless heat generation is controlled. Cracking and incipient disintegration, due to unequal contraction after expansion and to too rapid evaporation of uncombined water during early curing periods when excessive temperatures are permitted, demand that consideration be given heat control.

Rapid placing by modern methods lessens the opportunity for heat dissipation and the trend toward high early strength cements helps create a vicious circle wherein the higher temperature caused by the more rapid early chemical action in turn accelerates this action. High temperatures, caused by fine grinding, high lime or alumina content, are more evident in rich mixes and therefore enter into the design.

The heat rise in the average concrete mass will vary from 30 to 70 deg. F., within the first two weeks after placing depending upon the cement content, the thickness of cover and the ambient temperatures. Fig. 1 illustrates a typical temperature rise in mass concrete together with the theoretical rise that might be expected if no heat were lost. The cooling effect of the atmosphere upon concrete having less than ten feet of concrete cover is obvious. High initial heat in the concrete and high atmospheric temperatures raise the peak and decrease the rate of radiation. The behavior of the concrete during the cooling period following the peak temperature is influenced by the deferred hydration of the cement and, where radiation is slow, the return to normal temperature may require several years. An extensive research program disclosing the extent of such chemical action is an immediate need.

Control of heat generation involves mitigation of the causes coupled with engineering design that takes cognizance of the many influencing factors. Reducing the cement content in the mix may cut down the temperature rise as much as 40 per cent



EARLY TEMPERATURE RISE IN MASS CONCRETE

(although such a range is rarely possible in design). Changing cement grinding from 90 to 80 per cent fineness (per cent passing No. 200 sieve) may be made to curtail heat development 20 per cent (usually at the expense of ultimate strength). In manipulating the lime, alumina, magnesia, iron and other constituent parts of the cement to produce a slow setting concrete for mass placing, advantage can be taken of the fund of knowledge acquired in perfecting high early strength cements. The use of precast concrete blocks that can be cured under ideal moisture and temperature conditions and readily handled by cableways, offers a solution of the placing problem which rivals the common expedient of limiting the size of pours so as to facilitate radiation.

Readers are referred to the JOURNAL *for November, 1931, (Vol. 28), for further discussion which may develop. Such discussion should reach the Secretary by October 1, 1931.*

Tentative Specification for Ready-Mixed Concrete

*Report of Committee 504**

MILES N. CLAIR, AUTHOR CHAIRMAN

Committee 504 first presented at the February 1930 meeting a report[1] consisting principally of the work of the author-chairman. The specification included in that report received considerable helpful criticism and discussion. As the result of that discussion, both at the convention and at subsequent meetings of the Committee, a new specification was developed which was presented for your consideration in the November 1930 issue of the Journal. That specification was further modified by the committee at this convention, to meet the objection of some members of the committee and others interested. The specification proposed by this committee is in five sections. The first, General, logically deals with the limitation of the specification, definitions that are necessary for its understanding, and a general requirement in regard to the operation of the plant. The second deals with materials and refers generally to the usual specifications for materials. There is nothing startling nor unusual in either the first or second sections. The third section refers to concrete quality and proportioning, and this section is divided into five parts. Article A deals with concrete quality and proportioning generally, stating that they shall be in accord with the general specifications for concrete, plain and reinforced, but there is a modifying statement that the proportions may be changed at the discretion of the architect or engineer. The reason is that in the use of ready

*This report was presented by Mr. Clair at the 27th Annual Convention Milwaukee, February 24-26 1931, introducing the committee's recommendations for revision and tentative adoption of the specification published November, 1930; *Proceedings*, Vol. 27, p. 281. As amended and tentatively adopted by the convention, the specification follows Mr. Clair's introduction.—Editor.

[1]A. C I Journal, February, 1930; *Proceedings*, Vol. 26, p. 467.

mixed concrete, it is frequently necessary to make changes that might not be contemplated or necessary under the general specification. Article B deals with the measurements of the concrete aggregates and the cement by weight—by weight because of the necessity for ready handling of varied proportions. Central mixing or truck mixing plant operations must deal with different proportions at almost a moment's notice. However, we were faced with the fact that some operations use bag cement, so a footnote allows the use of bag measurements if you do not split bags. You can thus safeguard more or less the accuracy of your measurements, which brings it all back practically to a weight basis. The measurement of water may be by weight or by volume; that is in accordance with the usual practice. The volume of concrete produced by the batch shall be taken as the same absolute volume of the ingredients. That statement should appear in this specification, we feel, because it is frequently a matter of controversy, and it might as well be made definite. I think there is very little difference of opinion among engineers as to how the concrete should be measured, but there seems to be a good deal of difference among contractors. The next item deals with requirements for a statement of control quantities. The engineer wants to know what goes into every load of concrete, and so he should have a statement with each load as to what does go in. The next item requires that facilities be provided at the plant to enable the architect or the engineer to check at any time the quality of the materials and the accuracy of the measurements. That is just the same way as we have it at a steel plant. They have facilities at a steel plant so you can check up on the quality of the steel. The requirements for tests are the same as in the general specification. You will recall in the original specification the requirements as to tests were more stringent, but discussion has finally resulted in the requirement that they be no more stringent than required by the general specification. Section four deals with mixing and transporting concrete. This is the item which gave rise to the most discussion, because it is here that ready-mixed concrete differs most from ordinary concrete. I think however when we consider this item of the specification, we should keep in mind that we have been doing essentially the job of the ready-mixed concrete operation for many years on

our large jobs—large dam jobs and large road jobs. If that is kept in mind, we will not be so concerned about some of the limitations. We want trucks that do not lose any of the ingredients. Second, the concrete mixer shall be of the revolving type of mixer, whether operating on the truck or at a fixed point. We do not want to go through again, at this time, the evolution of the mixer so we are requiring that it be a revolving drum type of mixer. The committee felt that the limitations on the number of revolutions and the time of mixing, the capacity, that is, the load put into the mixer, should be in accordance with the manufacturers' rating and recommendation. That of course some day may be changed. Paragraph 3 of Article A, has involved considerable discussion. As originally written in the specification published last November it read:

(3) A maximum period of .... hours may elapse between the addition of the cement to the aggregates and the placement of the concrete in the forms; but the concrete as placed shall be of the consistency and workability required by the General Specification for Plain and Reinforced Concrete.

The committee, on reconsideration, felt that the emphasis was wrong and amended the paragraph to read as follows:

(3) The concrete as placed shall be of the consistency, workability, materials and water contents as required by the General Specification for Plain and Reinforced Concrete. In no case, however, shall the time elapsing between the addition of the cement to the aggregate and the placement of the concrete in the forms be more than .... hours[5] without special permission from the architect or engineer.

A footnote to this section is likewise modified. The original footnote read as follows:

The architect or engineer must indicate the limiting period. A maximum period of one hour is recommended.

The committee has voted that this footnote also be amended, to read as follows:

The Architect or Engineer must indicate the limiting period. An elapsed time of one hour has been found satisfactory under the usual operating conditions, but the permissible time may be longer under special conditions as determined by the workability and methods of placing.

A fourth item, is usual, that concrete shall not be retempered nor partially hardened concrete be placed in the structure. The fifth item requires precautions for the protection of concrete in transport when necessary.

Section B, Under Mixing and Transporting, deals with central plant-mixed concrete, provides that the concrete shall be mixed in accordance with the general specifications before being placed in transporting vehicle. In other words, you are not to count on the agitating effect of the usual agitating or non-agitating type of vehicle as being effective. The type of truck is limited to three classes—non-agitating, which is limited by the consistency of the concrete; the re-mixing type and the agitating type. The footnote under this section originally read:

> The Architect or Engineer must indicate the limiting slump. A maximum allowable slump of two and a half inches for non-agitating type truck and six inches for remixing type truck is recommended.

We felt that this, again, did not properly place the emphasis and the committee amended it to read as follows:

> The Architect or Engineer must indicate the limiting slump. An allowable slump of two and a half inches for non-agitating type trucks and six inches for remixing type trucks has been found satisfactory under usual operating conditions, but the use of greater slumps is permissible, if it can be shown by test that the concrete can be delivered without segregation.

That, incidentally is the second point concerning which there was the most discussion. The remaining part of the specification received much better treatment. Section C, dealing with Central Plant-Proportioned Truck Mixed Concrete included a precautionary paragraph that the procedure in loading the truck mixer shall be such that the cement and aggregate are uniformly distributed throughout the truck when the loading is completed. There is a requirement that the truck be so equipped that the amount of water added and the time of agitation or the number of revolutions during the transportation can be verified.

The fifth section of the specification deals with the requirement on delivery. The first provision is that the organization supplying concrete shall have sufficient plant capacity and transportation apparatus to insure continuous delivery at the rate desired; the second recommends that the interval between loads shall not exceed thirty minutes, or in any case be so great as to allow the concrete in place to become partially hardened; and third, that the method of delivering the concrete shall be such as will facilitate placing with the minimum of re-handling and without damage to the structure or concrete.

The committee has given considerable time to this specification; and has tried to lend a receptive ear to all suggestions. One suggestion only was received from outside the committee previous to today, since the publication of the specification in November. We feel it is worthy of acceptance as a tentative specification of the Society, although we recognize that the practice in this field is moving very rapidly and that modifications may be desirable before it is moved to a standard. I move, on behalf of Committee 504, the tentative adoption of the specification published in the November 1930 JOURNAL of this Institute, as here amended. (The specification as amended and tentatively adopted follows:)

TENTATIVE SPECIFICATION FOR READY-MIXED CONCRETE

*American Concrete Institute Tentative Specification 504-31-T**

MILES N. CLAIR, AUTHOR-CHAIRMAN

Committee 504, Specifications for Ready Mixed Concrete

1. GENERAL

(A) These specifications cover the special conditions affecting the use of Ready-Mixed Concrete and supplement the requirements of the General Specifications for Plain and Reinforced Concrete for this Contract.

(B) The term "Ready-Mixed Concrete" is used in these specifications as meaning[1] Central Plant-Mixed concrete and Central Plant-Proportioned Truck-Mixed concrete.

(C) The place where the materials are proportioned is referred to in these specifications as "the plant."

(D) The operation of the plant shall be under the direct supervision of a competent engineer.

2. MATERIALS

(A) All aggregates, cement and water shall be of the quality required by the General Specifications for Plain and Reinforced Concrete. All conveyors, bins and hoppers shall be cleaned of unapproved materials before starting to manufacture concrete for this contract.

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*Proposed by Committee 504, November, 1930, JOURNAL, p. 281, and now published with amendments adopted by the 27th Annual Convention, February, 1931.

[1]The Architect or Engineer must indicate which systems are allowed. It is recommended that in general both be included.

(B) No change shall be made in the materials used, during the course of this contract, except with the permission of the Architect or Engineer.[2]

3. CONCRETE QUALITY AND PROPORTIONING

(A) The concrete quality and proportions shall be in accordance with the requirements of the General Specifications for Plain and Reinforced Concrete. Proportions, however, shall be subject to change under the direction of the Architect or Engineer when necessary to produce proper workability or satisfactory strength.

(B) The measurement of the concrete aggregates and the cement[3] shall be by weight so that the proportions can be readily adjusted to assure a definite volume of concrete of the quality specified. The measurement of the water may be by weight or volume. The volume of concrete produced per batch shall be taken as the sum of the absolute volumes of the ingredients.

(C) A statement of the control quantities[4] used and the time at which the vehicle was loaded shall accompany each load of concrete. The organization supplying the ready-mixed concrete shall have available sufficient test data, certified by a competent concrete testing engineer, to enable the Architect or Engineer to determine whether the quantities used will give concrete of the desired quality and the stated quantity per batch.

(D) Facilities shall be provided at the plant to enable the Architect or Engineer to check easily at any time the quality of the materials and the accuracy of the measurements.

(E) Tests of the concrete as delivered to the forms shall be made during the progress of the work as required by the General Specifications for Plain and Reinforced Concrete.

4. MIXING AND TRANSPORTING CONCRETE

(A) *General*

(1) Transporting vehicles shall be so constructed as to insure rapid delivery without loss of ingredients.

(2) All concrete shall be mixed in a revolving drum type of mixer, whether stationary or movable. The mixing time or

[2]In the use of this specification for a particular project one of these must be stricken out throughout the text to avoid conflict.

[3]Bag measurement may be used provided the batches do not involve fractions of bags.

[4]"Control quantities" refers to the requirements of the specifications such as water-cement ratio, cement content, or proportions.

number of revolutions shall be as recommended by the manufacturer; but shall not be less than required by the General Specifications for Plain and Reinforced Concrete. The mixer shall not be loaded beyond the capacity recommended by the manufacturer.

(3) The concrete as placed shall be of the consistency, workability, materials and water contents as required by the General Specifications for Plain and Reinforced Concrete. In no case, however, shall the time elapsing between the addition of the cement to the aggregate and the placement of the concrete in the forms be more than 00 hours[5] without special permission from the Architect or Engineer.

(4) Under no circumstances shall partially hardened or retempered concrete be placed in the structure.

(5) Special precautions shall be taken to protect the materials from damage during transportation under extreme weather conditions.

(B) *Central Plant-Mixed Concrete*

(1) The Concrete shall be properly mixed in accordance with the requirements of the General Specifications for Plain and Reinforced Concrete before being loaded into the transporting vehicle.

(2) Non-agitating type transporting vehicles shall not be used to transport Central Plant-Mixed concrete when the consistency of the concrete is wetter than a —-inch slump.[6]

(3) Remixing type transporting vehicles[7] shall not be used to transport Central Plant-Mixed concrete when the consistency of the concrete is wetter than a —-inch slump.[6]

(4) Agitating type transporting vehicles may be used without restriction as to consistency.

(C) *Central Plant-Proportioned Truck-Mixed Concrete*

(1) The procedure in loading the truck mixer[8] shall be such

[5]The Architect or Engineer must indicate the limiting period. An interval of one hour has been found satisfactory under the usual operating conditions, but the permissible time may be longer under special conditions as determined by the workability and methods of placing.

[6]The Architect or Engineer must indicate the limiting slump. An allowable slump of two and a half inches for non-agitating type trucks and six inches for re-mixing type trucks has been found satisfactory under usual operating conditions, but the use of greater slumps is permissible, if it can be shown by tests that the concrete can be delivered without segregation.

[7]Re-mixing vehicles are those provided with special means of remixing or agitating the concrete to some degree during discharge but not during transportation.

[8]It is suggested that the cement and aggregates be fed simultaneously into the end loading type of truck mixer. For the top loading type of mixer this method of charging is not so essential; but is also recommended.

that the cement and aggregates are uniformly distributed throughout the batch when the loading is completed.

(2) Where the concrete is mixed during transportation of the batch to the job, means shall be provided so that the mixing time and the quantity of water added can be readily verified by the Architect or Engineer.

5. DELIVERY

(A) The organization supplying concrete shall have sufficient plant capacity and transporting apparatus to insure continuous delivery at the rate required.

(B) The interval between loads shall not exceed — minutes,[9] or in any case be so great as to allow the concrete in place to become partially hardened.

(C) The methods of delivering the concrete shall be such as will facilitate placing with the minimum of re-handling and without damage to the structure or concrete.

[9]The Architect or Engineer must indicate the interval allowed. A maximum of thirty minutes is recommended.

Tentative Construction Specification for Concrete Work on Ordinary Buildings (502-31-T)

*Report of Committee 502**

ARTHUR R. LORD, AUTHOR-CHAIRMAN

The report of Committee 502 was published in the Journal for November, 1929.[1] Discussions (published in March, 1930,[2]) prompted the Committee to make several revisions designed to cover all valid objections. As thus amended, by the Committee, the specification was tentatively adopted by the Institute by vote of the Convention at New Orleans a year ago.

In the early discussion the rather severe treatment accorded admixtures in this proposed standard went unchallenged. As time went on, however, I understand that this report began to cause the "Admixture Boys" (I borrow this convenient term from the discussion of one of them) some embarrassment. In considering their distress it is but fair to remember that the subject of admixtures has caused the Advisory Committee embarrassment also. Our long projected committee to report on admixtures has been the most difficult committee we have had to deal with in recent years. Only after long search and several tumbles have we found a man to head this committee who possessed the necessary freedom from prejudice, reputation for fairness and scientific accuracy, together with sufficient backbone to deal firmly with such experienced and persuasive gentlemen as the "Admixture Boys."

That was the situation when the Program Committee put this committee report on for discussion, with the committee proposing to ask that the tentative specification be sent to letter ballot for

*This report was presented by Mr. Lord at the 27th Annual Convention, Milwaukee, February 24-26, 1931.

[1]A. C. I. *Proceedings*, Vol. 26, p. 1.

[2]A. C. I. *Proceedings*, Vol. 26, p. 580.

adoption as a full standard. Since its tentative adoption three discussions brought up for the first time, the subject of the highly obnoxious Article 10. These discussions were published in the JOURNAL of September, 1930.[3] Fortified by an excellent dinner, stimulated by the hearty fellowship of last evening, the "Admixture Boys" decided to confer rather than to dance. Having reached without difficulty the unanimous conclusion that Article 10 was hostile and that something should be done about it at the eleventh hour, I was sent for. My personal safety was assured by the presence of Mr. Goldbeck as arbitrator, and I am pleased to recommend him for that service should any other group stand in a similar need.

I have never before seen so many admixture representatives peaceably assembled. If Article 10 serves no more useful purpose than to bring the representatives of all admixture manufacturers to a realization that their interests are mutual rather than opposing, that the very information that Committee 502 demands for the safe guidance of the users of admixtures is the thing most helpful to the admixture business, its brief existence will have been worth while. The meeting gave a free and full opportunity for frank discussion of the attitude of this committee—in which it is no less unanimous than the admixture representatives—towards this subject. After a few hours of friendly and hearty debate we all went into a huddle with the referee and as a result I am privileged to submit the "Goldbeck formula" for the consideration of the convention at this time.

The Goldbeck solution strikes out the first two paragraphs of the present Article 10 and substitutes one new paragraph.

Article 10 in the original publication of the proposed specification follows:

10. ADMIXTURES AND SPECIAL CEMENTS.

Nothing shall be added to the essential ingredients of concrete (portland cement, fine and coarse aggregate, and water) without the approval of the Architect or Engineer in writing and then only such materials shall be added as have been thoroughly tested by reputable independent investigators so as to demonstrate their effect on the strength, elastic properties, permeability and permanence of concrete produced from materials such as are being used on this work.

[3] A. C. I. *Proceedings*, Vol. 27, p. 99.

No special cement shall be used in place of standard portland cement except upon the same basis of adequate test and known effect upon all vital properties of the resulting concrete.

Whenever such admixtures or special cements are approved for use on this work in this manner they shall be used in accordance with the manufacturer's instructions and in conformity with their use in the tests on which their acceptance is based.

The paragraph which it is now proposed to substitute for the first two quoted above, follows:

Materials proposed for use in concrete in addition to the essential ingredients (portland cement, fine and coarse aggregate and water), or special cements proposed for use in place of standard portland cement, shall be permitted only with the approval of the Architect or Engineer in writing, on the basis of acceptable tests to demonstrate thoroughly the effect of such materials on the strength, elastic properties, permeability and permanence of the concrete.

The final paragraph of Article 10 is to stand.

The committee also wishes to make another change in this specification, designed to permit the use of cement in bulk as follows: At the end of the first sentence of the second paragraph of Article 5 add a new clause reading:

. . . except that cement may be shipped in bulk direct from manufacturers storage to this work, or to the central mixing plant where the concrete for this work is proportioned.

With these two changes I move you, Mr. Chairman, that tentative specification 502-30-T be approved as amended and be continued as a tentative specification of this Institute—502-31-T. While no arrangement was made to this effect last night—this morning, rather—I trust that some admixture representative will be gracious enough to second my motion.

He was, and the motion was carried without dissent.

Tentative Construction Specification for Concrete Work on the Small Job (506-31-T)

Report of Committee 506

ARTHUR R. LORD, AUTHOR-CHAIRMAN

Arthur R. Lord, author-chairman Committee 506, proposed, at the 27th Annual Convention, February 24-26, 1931, amendments to Articles 4 and 10 of Proposed Construction Specification for Concrete Work on the Small Job—first published as a report of the Committee in this Journal, September, 1930,[1] and discussed January, 1931.[2]

Article 4 in its original form and as amended is shown in the following—the words underlined to be deleted and italic indicating new wording:

4. Concrete materials

Portland cement shall be used for all concrete work. This cement shall be certified by the dealer to comply with all requirements of the standard specifications of the American Society for Testing Materials. (Serial designation: C9-26, or latest standard of this Society.)

Aggregate for this work shall be certified by the dealer (See Note 1 opposite)* to comply with the following specifications:

Fine aggregate for this work shall consist of well graded natural or artificial sand, or other approved inert materials with similar characteristics, taken from sources that have furnished satisfactory materials for previous concrete work for several years, or if from a new source shall be thoroughly tested for soundness and permanence. It shall comply with the provisions of the Tentative Purchase Specifications for Concrete Aggregates, E5-A-26T, *E5-A-29T*, (or latest standard of the Institute) of the American Concrete Institute. In applying these provisions the sieve analysis of dried aggregate shall show as follows:

Passing No. 4 sieve—not less than 85 per cent.

Passing No. 50 sieve—between 5 and 30 per cent. (Note 2 opposite.)

*References (see note opposite) are to the original printing of the specification where the "whys" of the provisions are discussed—Editor.

[1]A. C. I. *Proceedings*, Vol. 27, p. 65.

[2]A. C. I. *Proceedings*, Vol. 27, p. 525.

Passing No. 100 sieve—not more than 5 per cent.

Fineness Modulus—not less than 2.50

and the weight of clay and silt removed by decantation shall not exceed 3 per cent.

Coarse aggregate for this work shall consist of crushed stone, gravel, air-cooled blast-furnace slag, or other approved inert materials of similar characteristics, taken from sources that have furnished satisfactory materials for previous concrete work for several years or if from a new source shall be thoroughly tested for soundness and permanence. It shall comply with the provisions of the Tentative Purchase Specifications for Concrete Aggregates, E5-A-26-T *E5-A-29-T* (or latest standard of the Institute) of the American Concrete Institute. *The maximum size of the coarse aggregate shall be one inch unless the use of a larger maximum size is directed by the Architect or Engineer.* In applying these provisions the sieve analysis of dried aggregate shall show as follows:

Passing 1-inch sieve—not less than 95 per cent.

Passing No. 4 sieve—not more than 10 per cent.

Passing No. 8 sieve—not more than 5 per cent.

Fineness Modulus—between 6.30 and 7.50.

The weight of slag shall not be less than 65 lb. per cu. ft. The weight of soft, friable, thin, elongated or laminated pieces shall not exceed 3 per cent.

Aggregates shall be so stored and handled at all times as to remain separate from each other and free of all foreign material until placed in the mixer. (See Note 5.) Drainage shall be provided where necessary to avoid accumulation of water in aggregates.

Water shall be taken from the usual drinking supply and shall be paid for by this Contractor.

The same materials shall be used throughout the work, including the same brand of Portland Cement from the same mill, the same sand and gravel or crushed rock from the same source and from the same bin at the screening plant. (See Note 3.) This Contractor shall be fully responsible for the correct certification of all materials by the material dealer, as provided above. In case of doubt this Contractor shall pay for the necessary tests to be made under the direction of the Architect or Engineer.

To Article 10: "Measurement of the Batch" a new paragraph is added as follows:

When the concrete mix is determined by means of preliminary tests made by an approved laboratory and using the same materials as are to be used in the work, the water-cement ratio to be used in the work shall be taken as that giving a strength under standard laboratory conditions, at the age at which the concrete is to be put into service, not less than fifteen per cent greater than the ultimate strength required of the concrete in the work.

Both these amendments were accepted by the Convention and as thus amended the specification was tentatively adopted.

Tentative Specification for Supplying, Fabricating and Setting Reinforcing Steel on Ordinary Buildings (503-31-T)

Report of Committee 503

W. F. ZABRISKIE, AUTHOR-CHAIRMAN

The report of this committee published in this Journal, February, 1930;[1] formally presented at the 26th Annual Convention, New Orleans, February, 1930, and discussion published in this Journal, June, 1930,[2] was presented to the 27th Annual Convention, Milwaukee, February, 1931, for tentative adoption of the Proposed "Specification for Supplying, Fabricating and Setting Reinforcing Steel on Ordinary Buildings" with an appendix: "A Steel Setters Primer," motion by Mr. Zabriskie was seconded and adopted. The document now takes the serial designation as a tentative specification "503-31-T."

[1] A. C. I. *Proceedings*, Vol. 26, p. 444.
[2] A. C. I. *Proceedings*, Vol. 26, p. 910.

Concreting the Calderwood Tunnel*

BY W. R. JOHNSON†

THE PROJECT

The Calderwood hydro-electric-development, one of the projects of the Aluminium Company of America, is on the Little Tennessee river, approximately a mile above Calderwood, Tennessee.

The development consists of a 230-ft. thin-section-arch overflow dam, with a 40-ft. gravity dam several hundred feet below, built to form a pool to cushion the overflow from the main dam; a 26-ft. pressure tunnel 2400 ft. long and the power house. Two 56,000 h. p. turbines are installed to operate under a head of 213 ft. Fig. 1 shows the general layout of the development.

As this report deals primarily with the concreting of the tunnel only a brief description will be given of the operations leading up to this work.

The entire length of the tunnel including penstocks is approximately 2400 ft. It is circular for the first 219 ft. from the intake, with a diameter of 26 ft. 6 in. inside the 12-in. concrete lining. The first 131 ft. of this section is level and the remaining 88 ft. slopes downward at an angle of 51 deg. with the horizontal. The next section, 1727 ft. 6 in. slopes downward on a 2 per cent grade. This section is 26 ft. wide, with 11-ft. vertical side walls and a semi-circular crown of 13 ft. radius. The last 400 ft. branches out into three 16-ft. steel penstocks which slope steeply downward into the power house.

The entire tunnel was driven or excavated from an adit at the lower end of the tunnel. This adit is not a part of the permanent

*Presented by title (in author's absence) at 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.

†Concrete Technician, Calderwood Development, of the Aluminum Company of America.

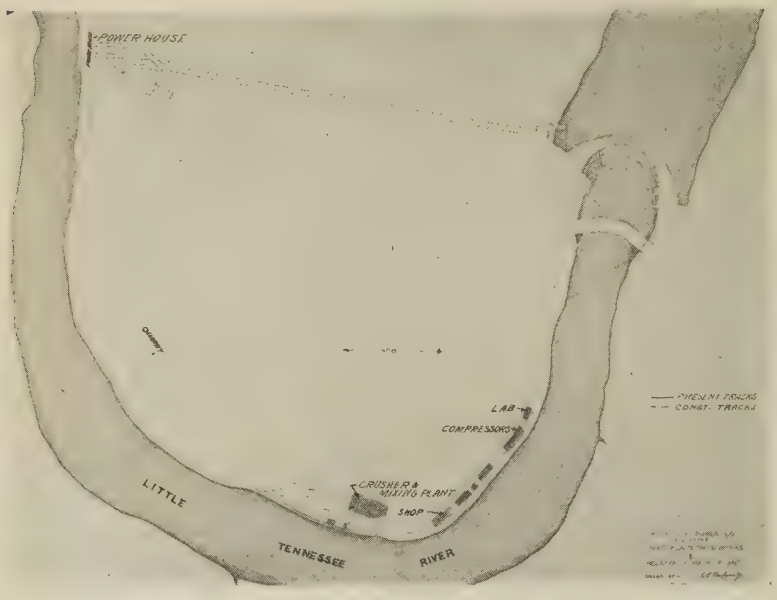


FIG. 1—GENERAL LAYOUT CALDERWOOD HYDRO-ELECTRIC DEVELOPMENT ON LITTLE TENNESSEE RIVER

tunnel and intercepts penstock No. 3 at the level of the main tunnel.

An 11 by 11-ft. heading was driven the entire length of the tunnel and later enlarged to full size by drilling and shooting. During the driving of the heading the muck was handled by an electric mucking machine (Fig. 4) and loaded into 2-ft. gage cars handled by storage battery locomotives. In the enlarging operations the muck was loaded into standard gage 6-yd. cars by a 1¼-yd. air shovel and removed by a gasoline-electric locomotive. The muck was later sent to the crushing plant and made into concrete aggregate, for use in the tunnel or dam as required. The dimensions of the tunnel are shown on Fig. 1 and 5.

All concrete was mixed at a plant located about ½ mile from the tunnel adit and transported to the tunnel in 2-cu. yd. sloping side buckets loaded on flat cars pulled by 20 ton "dinky" steam locomotives. The mixing plant was equipped with a 2-cu. yd. Smith tilting (56S) mixer.

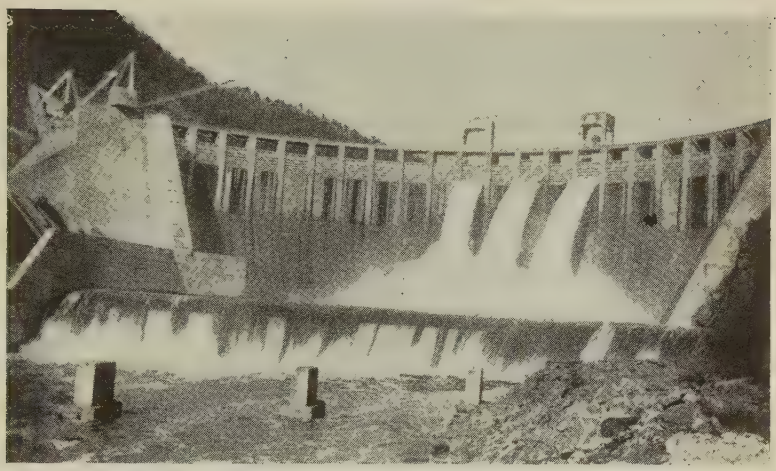


FIG. 2—CALDERWOOD DAM

CONCRETE MATERIALS AND TESTS

In addition to that obtained from excavations for the dam and tunnel, the aggregate used was taken from a quarry about 300 yds. from the crusher and mixer plant. This material was hard, dense arkose and made excellent aggregate. The coarse aggregate was graded from ¾ in. to 3 in. while the fine aggregate was manufactured and produced from the same material and graded from 0 to No. 4. Previous tests and experiments had shown that the elimination of material under ¾ in. in the coarse aggregate prevented raveling and separation of sizes in the bins, thus giving a more uniform product when but one bin was used for coarse aggregate.

The concrete for the entire Calderwood project was designed to produce a compressive strength of 3000 lb. per sq. in. at 28 days when tested as an 8 by 16-in. cylinder. All aggregate larger than 2 in. was removed from the concrete by hand picking at the time the specimens were molded. It was found, however, that a number of the cylinders could not be broken in the 200,000-lb. Olsen hydraulic testing machine, as the strength went beyond the capacity of the machine (4000 lbs. per sq. in. for an 8 by 16-in. cylinder). These strengths were being obtained from concrete placed in the main dam and with a cement content of 4½ bags

per cu. yd., which was considered the minimum that could be used to obtain the desired durability and water tightness in the structure of this type. The size of the test specimens was therefore reduced to a 6 by 12-in. cylinder with all aggregate larger

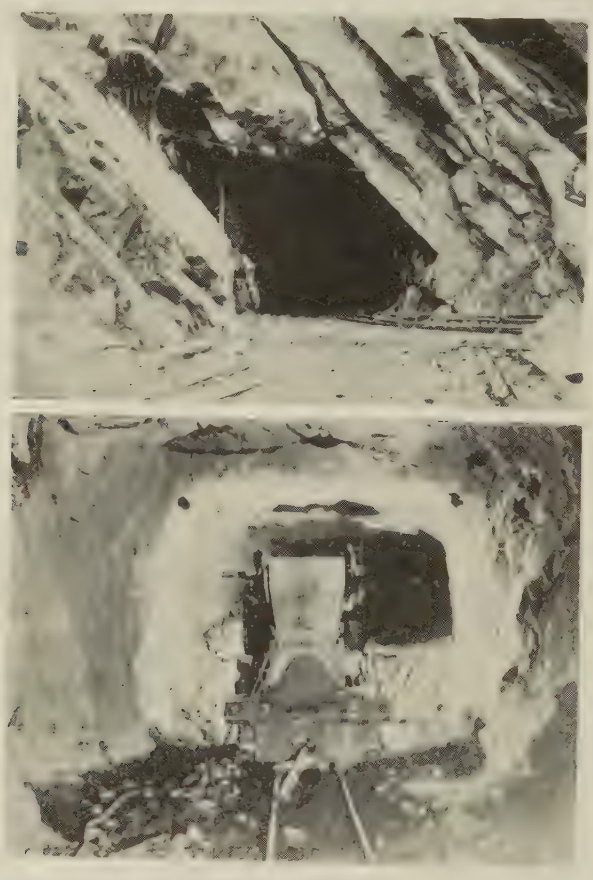


FIG. 3—TUNNEL CONSTRUCTION ADIT
FIG. 4—MUCKING MACHINE IN 11 BY 11 FT. HEADING

than 1½ in. removed from the concrete. Comparative tests showed that the 6 by 12-in. specimens gave strengths equal to 108 per cent of the strengths obtained from 8 by 16-in. specimens molded from the same batch. The average water-cement ratio

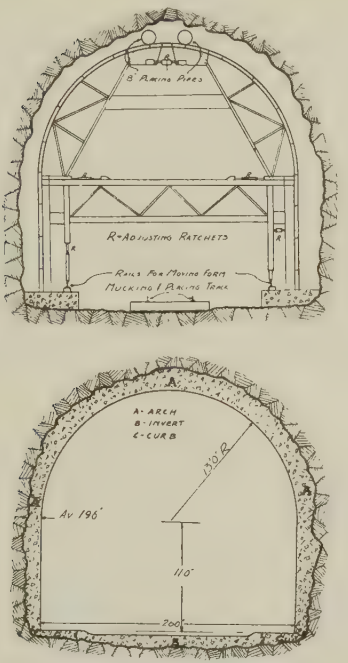


FIG. 5—MAIN TUNNEL SECTIONS, UPPER SECTION SHOWING STEEL FORMS

used for the concrete placed in the main dam was .95 (7.0 gal.), which gave an average strength of 4000 lb. per sq. in. at 28 days. For the tunnel concrete this water ratio was reduced slightly (.90) in order to give a greater factor of safety upon removal of forms 11 to 15 hours after placing. Preliminary tests made with 8 by 16-in. cylinders broken at the age of 14 hours indicated that a water-cement ratio of .90 (6¾ gal.) would give sufficient strength to move the 30-ft. Blaw Knox steel form without danger of any caving in the arch section. In actual practice the form was successfully moved several times, 11 hours after placing the concrete.

PLACING OPERATIONS

The total concrete placed in the entire tunnel, which included intake, main tunnel, and surge shaft, was 28,000 cu. yd. About 85 per cent of this yardage was placed with pneumatic placers.





FIG. 6—ELECTRIC VIBRATOR
FIG. 7—CONCRETE HOPPERS AND PNEUMATIC PLACERS

In general, three separate placing operations were used in the main tunnel. In order of procedure they were: curb, arch, and invert. The concrete sections of the 16-ft. circular penstocks were placed in two operations, bottom segment and main segment, or sides and top. Wooden forms, built in place, were used. The incline section of the main tunnel, which was circular, having a diameter of 26 ft. 6 in., was placed in the same manner as the penstocks. The surge shaft, which had a diameter of 18 ft. and an angle of 51 deg. with the horizontal, was placed in one operation, using collapsible, movable, wooden forms.

In the concreting of the main tunnel flat cars of standard gauge, mounting specially designed hoppers, were used for hauling material from the construction adit or portal into the tunnel. Each car was equipped with four hoppers (Fig. 7), each hopper having a capacity of two cubic yards and arranged so that concrete could be dumped simultaneously from both sides of the car into the curb forms or pneumatic guns, as the case might be.

The placing of the concrete in the main section was started at the foot of the riser, upstream, and progressed down hill on a 2 per cent grade. The curb (Fig. 5) for the entire length of the tunnel was placed first and later used as a foundation for moving and setting the 30-ft. collapsible Blaw Knox steel form used in concreting the arch section. The invert, or tunnel floor, was placed with one pneumatic gun, the operations following 60 to 90 ft. behind the arch placement.

Main Tunnel

Curb—The foundation or bottom was thoroughly cleaned of all muck and washed with a hose. The hopper cars, pulled by a gas electric locomotive, then brought the concrete from the tunnel portal to the forms where it was allowed to flow by gravity from the bottom gate of the hopper through a short, movable chute into the forms where it was spaded into place. Two sections of curb, one on each side of the tunnel, were placed simultaneously.

Arch—The average cycle of operations for concreting of a 30-ft. section of arch was as follows:

1. Moving and setting forms, 3 hours.
2. Placing concrete, 6 hours.
3. Setting and hardening of concrete, 15 hours.

This schedule was adhered to as closely as possible. Delays of different types, such as breakdowns of pneumatic guns, derailment of hopper cars, placing pipe troubles, etc., prevented placing every day. The average loss of time due to these troubles, however, was only about one day every two weeks. The first section of arch placed required 36 hours. Within 10 days this time had been cut down to 6 hours, which was the average time required for placing, although several sections were placed in less than 5 hours.

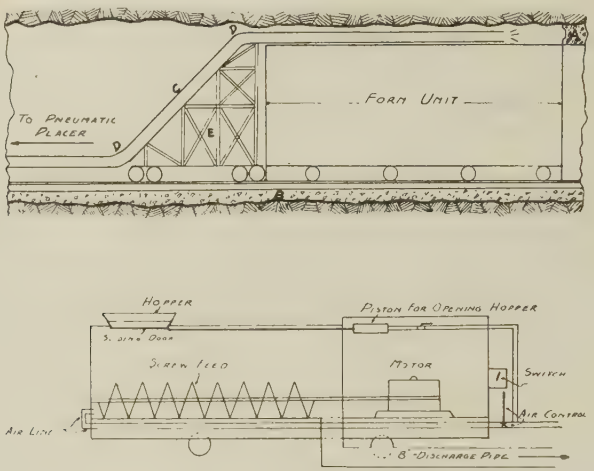


FIG. 8—(UPPER) PIPE SECTION SHOWING PLACING METHOD

LEGEND

A—PREVIOUS POUR
B—COMPLETED CURB
C—8 IN. PLACING PIPE
D—45 DEGREE BENDS
E—PIPE SUPPORT ON STANDARD FLAT CAR

(LOWER) RANSOME PNEUMATIC PLACER

Fig. 8 shows the set-up used for placing concrete in the arch. The procedure in filling the forms was as follows:

The concrete was brought from the construction adit to the form in hopper cars drawn by a gas-electric locomotive. From the hopper it was fed by gravity through the gates at the bottom into the two pneumatic concrete placers, one on each side of the car. These placers had a capacity of 14 cu. ft., and were of the horizontal type. The placers operated as follows:

The concrete entered the guns (Fig. 8) through a sliding door controlled by compressed air and at the back end and on top of the cylindrical barrel. From here it was forced by a screw feed, operated intermittently, away from the sliding door toward the front end of the barrel until the gun was fully charged. The sliding door was then closed and the screw feed and the air turned on. This action forced the concrete into the 8-in. discharge pipe through which it was forced by the air to its final place of deposit in the forms.

Attached to the forms with clamps were two electric tampers (Fig. 6), one on either side. These were run intermittently during the placing operations and were, no doubt, responsible to some extent for the smooth surface obtained.

In some of the first placements difficulty was experienced with honeycomb occurring in the lower corners next to the previous day's pour. This was caused by segregation in the delivery pipe during the blowing operation. The separation was such that the coarse aggregate was always delivered ahead of the mortar to an extent depending on the plasticity or workability of the mix.

To overcome this difficulty a batch of stiff mortar was first "shot" into each of the lower corners. This was followed by concrete in which the amount of coarse aggregate had been considerably reduced. About the first 15 per cent of the form was placed in this manner, after which the stone content of the concrete was brought back to normal. Very little honeycomb occurred after this method was adopted.

The two major problems encountered in the concreting operations were placing pipe breakdowns and improper consistency of the concrete. The second condition was in most cases the result of the first. The majority of the pipe troubles occurred at the elbows or bends, and the replacements of these sections occurred frequently. The first bends used were made of extra-heavy steel pipe. At the upper elbow, next to the form (Fig. 8), this type of bend was not good for placing more than 150 cu. yds. of concrete. The lower bends, next to the gun, lasted considerably longer. The second type tried was a Hagen-White iron elbow with reversible back. This elbow was a great deal more successful, and, by reversing the back, was good for placing approximately 600 cu. yds. of concrete, after which the backs were replaced.

Trouble was also encountered with pipe flanges, three different types being used before finding one that was satisfactory. These flanges and the order in which they were tried follow: cast-iron, ferro-steel, and steel.

The ends of the discharge pipes were equipped with 22½ deg. extra-heavy steel bends. Frequent patching was necessary to keep these in shape. Some 45 deg. bends were tried and also some

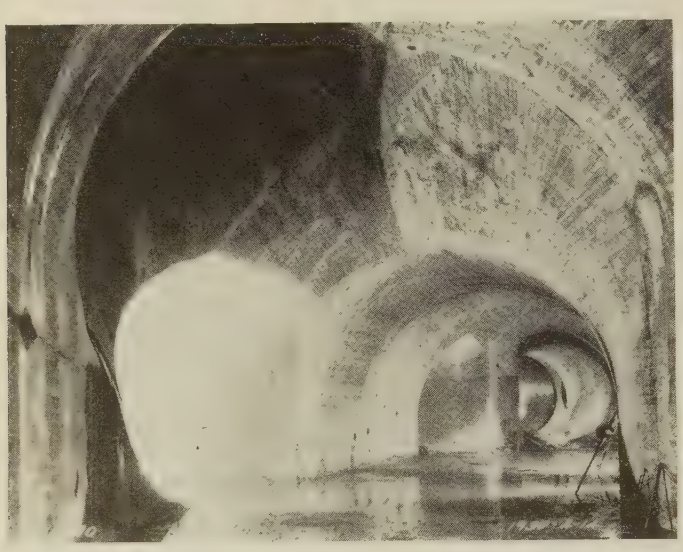


FIG. 9—MANIFOLD SECTION OF TUNNEL SHOWING ENTRANCES TO PENSTOCKS AND SURGE SHAFTS

straight sections. The most satisfactory position for the ends of the discharge pipe at the beginning of a pour was 5 to 6 ft. from the back of the bulkhead. After the form was one third to one half full, however, the position of the end of the discharge pipe or whether it was equipped with a bend or was straight did not seem to matter, equally good results being obtained with the end of the pipe placed 20 ft. from the bulkhead.

The consistency or workability of the concrete was one of the controlling factors in the placing operations, and great care was exercised to keep it correct in order to avoid delays in placing, as well as to prevent honeycomb in the finished job.

For proper placing, it was found that the concrete should be neither too wet nor too dry. A mix that was too wet would segregate badly during placing operations, while one that was too dry could be removed from the hoppers only with great difficulty. The ideal mix for pneumatic placing with materials used on this job was one that gave a slump of 4 to 5 in. with approximately a 1:2:3 mix and which did not segregate upon

being removed from the hopper cars. A mix of this type would "shoot" with great ease and segregate but little in the form.

As short a delay as 15 minutes at the placers during concreting operations would be sufficient to upset the entire cycle of operations, so that the ultimate delay might easily total an hour or an hour and a half before all of the setting concrete could be removed from the loaded hopper cars and buckets. Having no means with which to keep the concrete in the buckets or hoppers agitated when allowed to stand for 30 minutes or more, great difficulty was experienced in removing it, especially from the hoppers, after this time. To prevent delays of this type as much as possible, the inspector kept in constant touch with the mixer plant and the transfer operations at the tunnel entrance. No concrete was transferred from the buckets to the hopper cars if a breakdown occurred, and if the delay amounted to more than 30 minutes, the material was sent to the dam where it could be easily placed in the open forms by means of the electric tampers used in this work.

Invert—After thoroughly cleaning the invert, or bottom, of all muck and dirt, and washing with a hose, the concrete was brought in a hopper car to the placer on the right hand side of the tunnel. From here it was blown through an 8-in. pipe, 100 to 125 ft. long, into the invert form which was a great deal like a small section of pavement slab. A small wooden bulkhead, or stop, was used at the discharge end of the placing pipe to prevent the concrete from scattering unduly. From this point the material was shoveled into place and brought to grade by means of a screed, after which the surface was finished by means of a wooden float.

While this method of placing produced some surface laitance, due to the wet mix necessarily used with the pneumatic placers, it was decidedly the most economical process. The set-up was the same as that used in placing the arch section with the exception of one or two small changes in pipe connections. The work was done at night, thus not interfering in any way with the placing of the arch section.

With the small temperature changes encountered in a tunnel of this type, with no alternate freezing and thawing, and with

practically no abrasive or wearing action due to the slowly moving water, the writer does not believe that scaling on account of this laitance will be particularly detrimental to the concrete lining.



FIG. 10—MAIN TUNNEL

Penstocks and Riser

The same general procedure was used for concreting the riser and penstocks as was used in the main tunnel, with the exception that the forms were wood, built in place. Due to the small size of the penstocks, but one placer was used. This made it necessary to shift the end of the delivery pipe from one side to the other. Vibrators were not used, the concrete being booted and tramped into place.

Surge Shaft

The two-yard buckets of concrete were removed from the train and dumped into a hopper mounted on a platform at the foot of the mountain in which the surge chamber and shaft were located. From here it flowed by gravity into a second hopper mounted on a specially built car which was then pulled by cable, powered from the unloading derrick, approximately 250 ft. up a steeply inclined narrow gauge track to a third hopper on the mountain side. After being dumped into this hopper, the concrete flowed through a short chute into an upright pneumatic placer of somewhat different type from those used in the tunnel. This placer then "shot" the material through an 8-in. pipe, approximately 200 ft. long, into a fourth hopper at the top of the construction adit of the surge shaft on the mountain side. The concrete then flowed by gravity from this hopper through a second 8-in. pipe and into the surge shaft forms, its final place of deposit.

The somewhat complicated method described above was adopted in order to utilize a part of the same set-up that had been used previously in placing the power house protection walls.

STRENGTH TESTS

In Fig. 11 are plotted the 28 day strengths obtained from the concrete placed in the main tunnel section. The curves show that the average strength was approximately 4000 lb. per sq. in. The average mix was 1:2:3 by dry loose volumes, and the average cement content 1.5 bbls. (6 bags) per cu. yd. of concrete. The surge shaft and penstock concrete gave approximately the same result.

GROUTING

Several months after the completion of the concreting operations the entire tunnel was grouted in order to give it additional strength and water tightness. In the roof of the arch section of the main tunnel, where the cavities were comparatively large, a rich mortar was used for this purpose, while in the other sections a neat cement mixture was used. The grout, or mortar, was forced into place through a hose by means of a grout pump or mixer, capable of exerting pressures up to 100 lb. per sq. in. Holes drilled at regular intervals through the concrete lining in the roof of the tunnel provided the necessary connections for the hose for forcing the grout into place (Fig. 10).

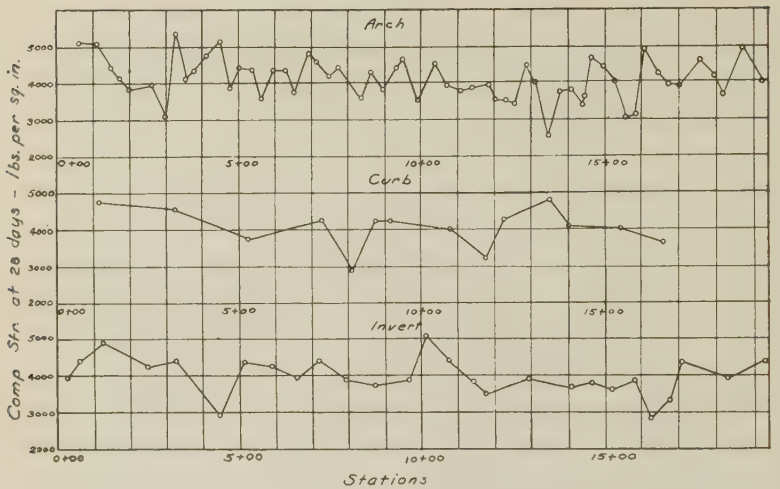


FIG. 11—TUNNEL CONCRETE STRENGTHS AT 28 DAYS

CONCLUSION

The success of a concreting operation using the methods described is dependent in a large degree on the proper co-ordination and co-operation of the different units involved. Without this co-operation costly delays may occur to upset the entire cycle of operations, thus running the costs up appreciably.

Acknowledgment is therefore made to those in responsible charge of this work for their co-operation with the writer in obtaining the results they did. The writer also wishes to thank in particular R. F. Taylor and E. B. Rayburn who, in their capacity as inspectors, worked tirelessly to obtain a high quality of concrete and its proper placement in the finished job.

Calderwood Development was designed under the direction of James W. Rickey, Chief Hydraulic Engineer of Aluminum Company of America, by James P. Growdon, Principal Assistant Hydraulic Engineer. I. G. Calderwood was the General Superintendent of Construction, under whose direction the tunnel was built.

Readers are referred to the JOURNAL *for December, 1931, (Vol. 28) for further discussion which may develop. Such discussion should reach the Secretary by Nov. 1, 1931.*

Field Practices in Using Concrete Aggregates In Multiple Sizes*

BY T. C. THEE†

The division of Management of the United States Bureau of Public Roads is entering its eighth year of study of highway production problems and of the methods of eliminating waste in the construction of highways.

Studies of construction methods and equipment are conducted in all parts of the United States and include nearly all types of highway construction projects, including portland cement concrete, asphaltic concrete, sheet and sand asphalt, bituminous macadam and brick pavement. The grading work includes power shovels, elevating graders, wheeler and fresno outfits. Miscellaneous studies include drilling and blasting operations, aggregate crushing plants, local pit plants, etc.

Field investigational work is in constant operation to study possible changes that can be recommended further to reduce cost without sacrifice of quality. We hold quality of work as paramount. In these attempts to reduce costs and at the same time maintain quality we feel that we have made steady progress. The studies are cooperative, in which the Bureau, State and County highway officials are all concerned.

Coarse aggregate in separated sizes is now being used by the states to some extent where the maximum size is 1½-in. or more, or where a foreign aggregate is brought in to be incorporated in the mix to meet grading specifications.

Three states, California, Washington and New Jersey, require coarse aggregate to be batched in two sizes under all conditions. North Carolina, Louisiana, Georgia, Arkansas, Maine and Vermont require two-way batching under certain conditions.

*Presented at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931, with discussions by Messrs. Hambrecht and Langley, which follow in these pages.

†Assistant Highway Engineer, Division of Management, Bureau of Public Roads.

A few other states have used, or are using, two-way batching on special projects. Wisconsin tried three sizes of coarse aggregate on special projects last year, and on one project four sizes. No state requires a three-way split in batching. One state tried three sizes but has returned to the use of two sizes of coarse aggregate.

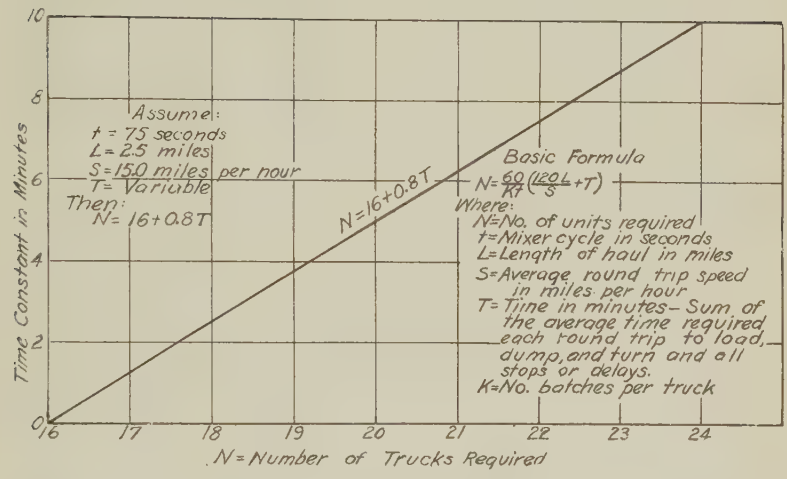
This separation of sizes is to eliminate as much as possible the segregation that takes place in car delivery and in stock piling the material, so as to maintain uniform gradation of the coarse aggregates and to reduce the batch-to-batch variation.

In using separated sizes of coarse aggregate the states specify the grading for each size. The point where the separation should be made is stated in the range of size. For example, No. 1 aggregate would be from 2½ in. to 1 in. and No. 2 aggregate from 1 in. to ¼-in. The material is delivered by rail in these two sizes. Two separate stock piles are maintained for these and one stock pile for the sand. Modern three compartment bins should be used and the three aggregates weighed separately. Moisture adjustments are made on the separate sizes and on the sand. The percentage of each size of coarse aggregate is adjusted to meet specifications and to obtain the least void content in relation to the best gradation and economy of aggregate production.

From the standpoint of batcher operation there is but little objection to two-size batching of the coarse aggregate where three-compartment bins are available. The real difficulties come in finding room for three instead of two stock piles, and in getting cars spotted correctly, as well as having each shipment carry the proper amounts of each size and the proper gradation.

Delays sometimes occur also at the beginning of a job or even in the progress of a job, due to variations at the producer's plant in the gradation of each size of the specified aggregates. As a consequence, the mix is often changed and this again may result in delays to the contractor because of the difficulty in ordering accurately the proper amount of each size.

These are all matters of real importance which cannot be forecast with precision. They do add to the difficulties with which the contractor has to contend and increase the opportunities for delay.



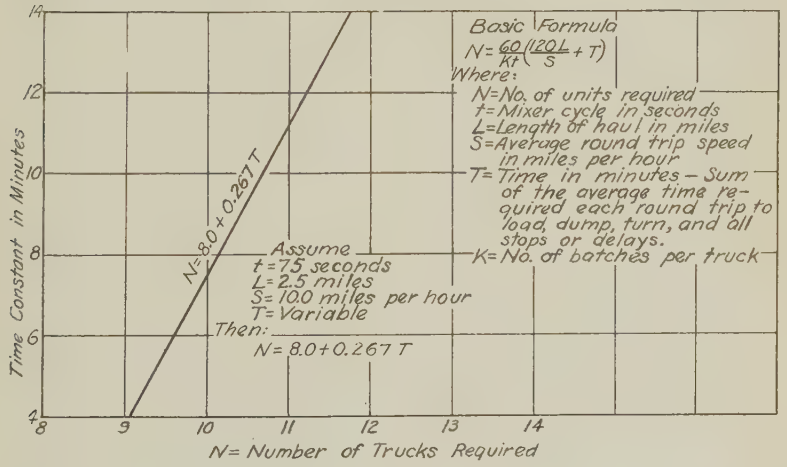


FIG. 1—EFFECT OF TIME CONSTANT ON THE NUMBER OF ONE BATCH TRUCKS REQUIRED ON CONCRETE PAVING

FIG. 2—EFFECT OF TIME CONSTANT ON THE NUMBER OF THREE BATCH TRUCKS REQUIRED ON CONCRETE PAVING

With the modern three-compartment batcher the time required for handling the three-way batch (two sizes of coarse aggregate and one of sand) is very little greater than for the two-way (one size of coarse aggregate and one of sand). Either type can be loaded while the truck is driving in, and the batch dropped in from 10 to 15 seconds, which is the length of the necessary stop for a one-batch truck. Even a three-batch truck is readily loaded with the three batches of sand and two sizes of stone or gravel in 75 seconds.

Where two bins are used the truck "time constant" is of course increased by the time required to get in position at the second bin and receive the batch or batches of the material handled by the second bin. A second bin reduces the crane operating time, through time lost in moving the crane from stock pile to stock pile and bin to bin. If the yard layout is poor, the time lost in moving the crane and the truck "time constant" may be considerable. This loss may be as much as three minutes for a three-batch truck, necessitating one or two extra trucks on the average job. This item alone is costly. Figs. 1 and 2 show the effect of time constants on the number of trucks required.

Stock piling practice varies so greatly and the requirements imposed by the states are so different that really comparable data are few. Under average conditions, when loading from cars to bins the crane cycle seems to vary between 30 and 45 seconds, depending on the skill of the operator, the angle of swing, and the general working conditions. When the car is nearly empty the rate usually is considerably slower. The fastest rate is usually obtained in working from a full car to stock pile, when if the swing is short the cycle may be as low as 25 seconds. The rate of operation does not seem to be appreciably affected by the size of the crane. That is, of two cranes in equally good condition the larger may be expected to operate on about the same cycle as the smaller. This seems to hold good at least for sizes from ¾ to 1½ cu. yd.

The most efficient contractors are finding it difficult to unload cars, stock pile, and fill the bins during the day and have to resort to several hours operation of the crane at night.

On the basis of a 60-second mixing time, a 30-cu. ft. batch, averaging 45 batches per hour using five sacks of cement per

cu. yd. of concrete, and a workability factor b/b_o of 0.76, 63,900 pounds of sand and 114,300 pounds of gravel will supply the 27-E paver per hour or a total of about 60 cu. yd. of material per hour.

With a crane averaging a cubic yard of material per bucket with a 35-second cycle, approximately 90 cu. yd. of material would be handled by the crane per hour, with allowance for necessary delays. This means that under favorable conditions the crane can produce about 50 per cent more than the requirements of the mixer if the material is loaded directly from cars to bin. If the material has to be stock piled first, then the crane would have to operate unloading cars partly at night. If material is delivered by trucks either from a nearby commercial plant or a local plant, the crane can easily maintain the necessary supply within the regular hours.

On the basis of a 50-second mixing time with a 33-cu. ft. batch, the crane will have available approximately 18 per cent leeway to load from cars to bins direct. However, if the specifications are such that two bins must be used, using three sizes of coarse aggregate, two cranes will have to be employed.

Where three sizes of coarse aggregate are delivered to a job by rail and three-batch trucks are used, the requirement of three sizes of coarse aggregate will cost the state indirectly through the

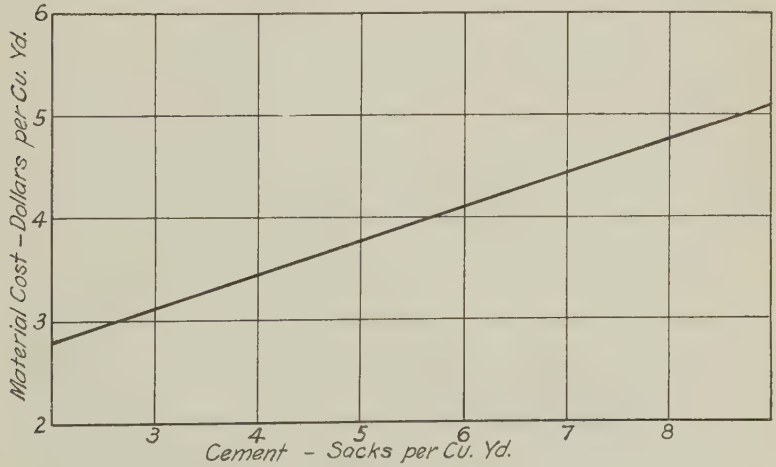


FIG. 3—EFFECT OF CEMENT CONTENT ON COST OF CONCRETE

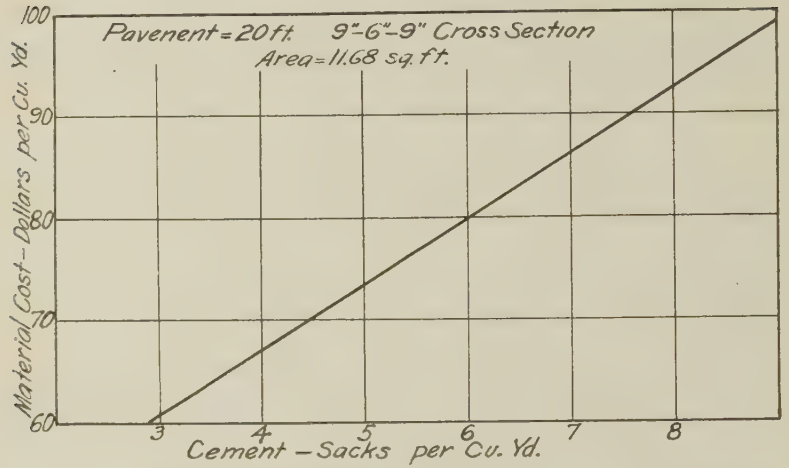


FIG. 4—EFFECT OF CEMENT CONTENT ON COST OF CONCRETE

contractor about $105 per day, or an average of $630 per mile, or 5.4 cents per sq. yd. of pavement. This is a cost equivalent of adding 0.78 sacks of cement to a 33-cu. ft. batch. Fig. 3 and 4 show the effect of cement content on the cost of pavement per cu. yd. and per sq. yd. respectively.

With a local or commercial plant delivering by trucks, the three sizes of coarse aggregate will cost approximately 1½ to 2 cents per square yard more than two sizes. No accurate average costs are available showing how much more three sizes of coarse aggregate cost to produce over one or two sizes.

Two projects were selected in Sheboygan county, Wisconsin, for special study of the use of multiple sizes of coarse aggregate and to investigate the effect of mixing time and batch size on the uniformity of ingredients of the concrete produced by 27-E pavers, and on the uniformity of resulting strength of concrete both for compression and transverse strength. The associated unit costs were also collected and studied. Before these two jobs were advertised to bidders careful laboratory experiments were conducted by the Wisconsin Highway Department to determine the desired grading and mix to use with the different sources of supply of aggregate materials. No discrimination was made between the different brands of cement available, although some difference existed.

On these two Wisconsin projects—S. A. P. 2916 and S. A. P. 2926—three sizes of coarse aggregate were specified with definite grading requirements. Five sacks of cement per cu. yd. of concrete as the minimum and six gal. of water per sack of cement as the maximum, were recommended so as to produce at least 650 lbs. per sq. in. transverse strength at 28 days.

The mixes on both jobs were designed for the minimum allowable cement content and maximum allowable water content. The workability of the concrete was maintained through the adjustment of the gravel and sand content employing the workability factor b/b_o as developed by Richart and Talbot. The method is shown graphically and the process of establishing the mix can be interpreted from the chart, Fig. 5. A workability factor b/b_o ratio of 0.76 seemed to work best on both jobs. The dashed line on the chart shows the mix developed on the basis of a b/b_o ratio value of 0.76.

The three sizes of aggregates were also recombined experimentally using different percentages of each size, in order to obtain as low a void content as possible without sacrifice of gradation in obtaining workable concrete.

The proportions of the various sizes as required by the specifications are as follows for S. A. P. 2926 (Sheboygan county, Wisc.).

No. 1	2½ in. to 1½ in.	30-35 per cent
No. 2	1½ in. to ¾ in.	45-50 per cent
No. 3	¾ in. to ¼ in.	15-25 per cent

No. 1 Coarse Aggregate, 2½ in.-1½ in.

	Per Cent
Retained on 3 in. circular opening	0
Retained on 2½ in. circular opening	0- 10
Retained on 1½ in. circular opening	90-100
Retained on 1¼ in. circular opening	100

No. 2 Coarse Aggregate, 1½ in.-¾ in.

	Per Cent
Retained on 1¾ in. circular opening	0
Retained on 1½ in. circular opening	0- 10
Retained on ¾ in. circular opening	90-100
Retained on ½ in. circular opening	100

No. 3 Coarse Aggregate, ¾ in.-¼ in.

	Per Cent
Retained on 1 in. circular opening	0
Retained on ¾ in. circular opening	0- 5
Retained on ¼ in. circular opening	95-100
Retained on No. 10 sieve	100

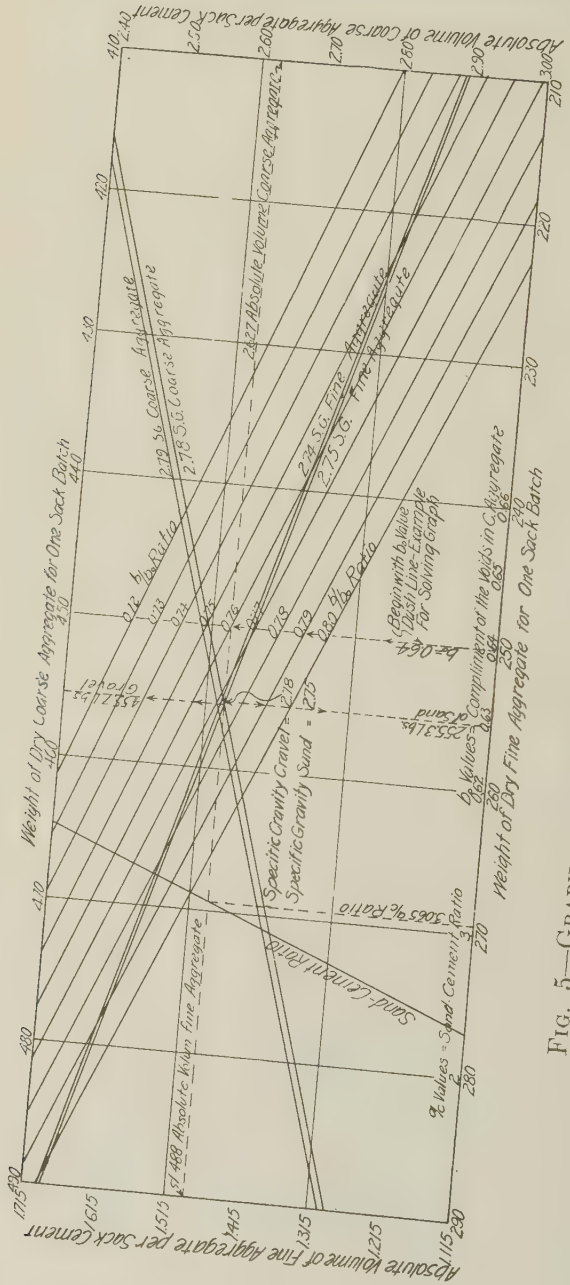


FIG. 5—GRAPH FOR AGGREGATE ADJUSTMENT IN MAINTAINING WORKABILITY WITH FIVE SACKS CEMENT PER CUBIC YARD OF CONCRETE AND 6 GALLONS OF WATER PER SACK OF CEMENT

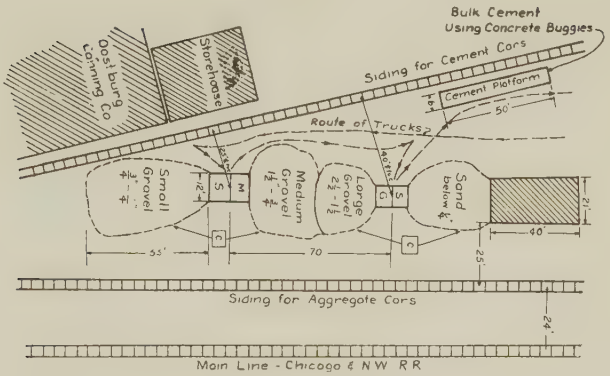


FIG. 6—PLANT LAYOUT, WISCONSIN S. A. P. 2916, SHEBOYGAN COUNTY—THREE SIZES OF COARSE AGGREGATE

Figs. 6 and 7 show the plant layout and set-up on Wisconsin S. A. P. 2916. It is obvious that a very crowded condition existed. Using three sizes of coarse aggregate made it very cumbersome. The plant was such that trucks lost on an average of two minutes over a typical set-up, resulting in the use of one and sometimes two trucks more than would normally be used; an extra crane was required; the material was delivered by rail.



FIG. 7—PLANT SET-UP, S. A. P. 2916, SHEBOYGAN COUNTY, WISCONSIN, USING THREE SIZES OF COARSE AGGREGATE AND ONE OF SAND; MATERIALS DELIVERED BY RAIL

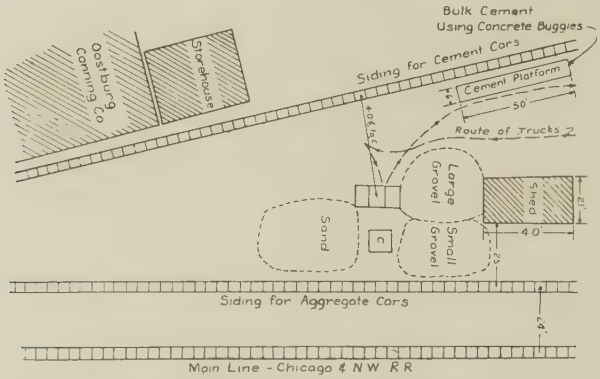


FIG. 8—SUGGESTED PLANT LAYOUT, WISCONSIN S. A. P. 2916, SHEBOYGAN COUNTY—TWO SIZES OF COARSE AGGREGATE

Fig. 8 shows the same plant site but with equipment and stock piles planned for handling two sizes of coarse aggregate. This set-up would not result in any greater cost than for only one size of coarse aggregate.

Fig. 9 shows the plant on one set-up on Wisconsin S. A. P. 2926, showing the number of turns that were required by the trucks on this job.

Fig. 10 shows the same set-up with a recommended change that could be used on similar jobs. Turning time for the trucks will be eliminated.

If but two sizes of coarse aggregate were used a three-compartment bin could have been used with one stop for the trucks. This would have been a most economical set-up and still have utilized the advantage of multiple size aggregates.

On S. A. P. 2926 the material was delivered by truck from a local pit and only one crane was necessary to supply the mixer when a 33-cu. ft. batch and a 50-sec. mixing time were used, with an average of fifty-five 33-cu. ft. batches an hour. This means that the crane handled 238,931 lbs. of sand and gravel per hour average, or about 80 cu. yds. of material per hour.

Fig. 15 shows a set-up observed this summer handled by one crane and straight-line loading method. Bulk cement was placed in the batch between the stone and sand which eliminated cover-

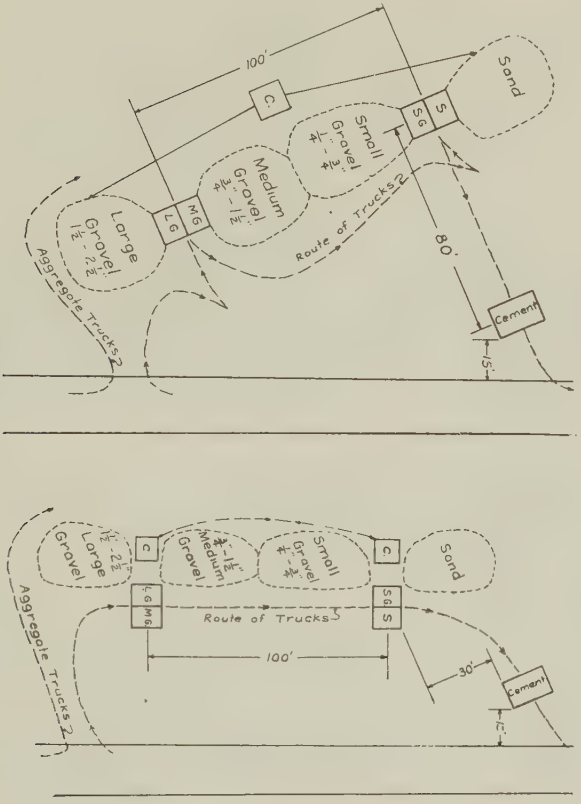
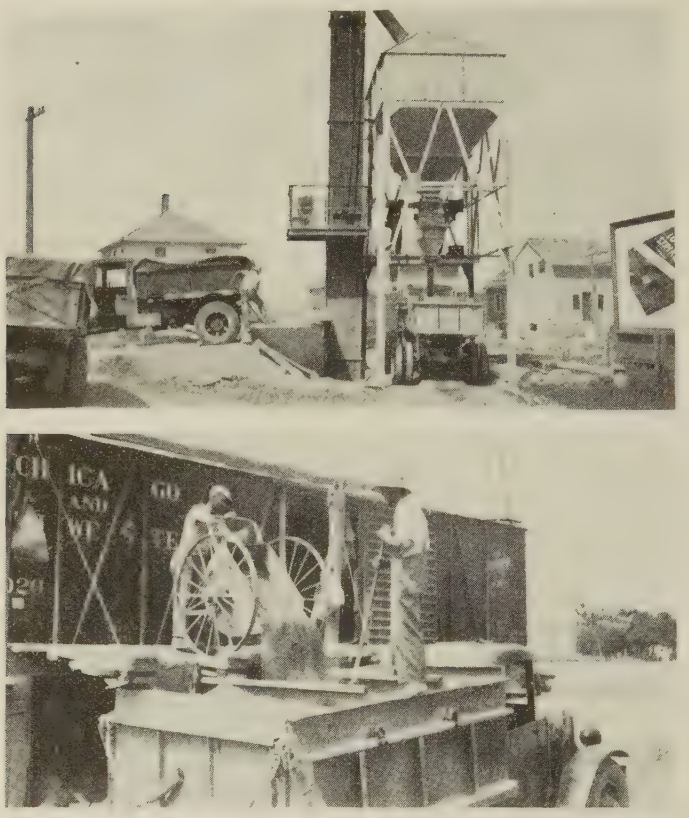


FIG. 9—PLANT LAYOUT, WISCONSIN S. A. P. 2926, SHEBOYGAN COUNTY—THREE SIZES OF COARSE AGGREGATE

FIG. 10—SUGGESTED PLANT LAYOUT, WISCONSIN S. A. P. 2926, SHEBOYGAN COUNTY—THREE SIZES OF COARSE AGGREGATE

ing the cement. This method proved very satisfactory. It eliminated having cement dumped into the skip first and choking the throat of the skip so as to retard material entering the drum. This method also eliminates the cement hitting the blades and bucket of a mixer drum causing them to become badly coated and reduce their efficiency. Some engineers prefer to have the order of loading changed to second size aggregate, sand, cement and then coarse aggregate.



FIGS. 11 AND 12—METHODS OF HANDLING CEMENT IN BULK ON S. A. P. 2916 AND S. A. P. 2926, SHEBOYGAN COUNTY, WISCONSIN

Incidentally, this contractor was using sack cement and obtained permission to try bulk cement, at a saving of eleven cents a barrel. Use of cement in bulk assures the proper weight of cement, enables a contractor to maintain a constant maximum batch and eliminates the cost of handling empty sacks. The cement retained in the sacks will average 0.4 lb. per sack. A state buying cement can make a large saving in the last mentioned item alone. As its advantages become more generally appreciated, it is believed the use of bulk cement will be extended. Up to the present time the most popular way of handling cement



FIG. 13—TYPICAL PLANT SET-UP, S. A. P. 2926, SHEBOYGAN COUNTY, WISCONSIN. MATERIAL HAULED BY TRUCK FROM LOCAL PIT—COARSE AGGREGATE IN THREE SIZES

FIG. 14—LOCAL PIT PLANT LAYOUT EQUIPPED FOR THREE SIZES OF COARSE AGGREGATE FOR S. A. P. 2926, SHEBOYGAN COUNTY, WISCONSIN

in bulk at the batcher plant is with two-wheeled concrete buggies. Two cars of cement are unloaded at the same time. A wooden platform is constructed to the same elevation as the box car floor and with a length sufficient to reach the doors of both cars. Usually two men are used in each car loading the buggies, two men to wheel the buggies to a platform scale, two men to weigh

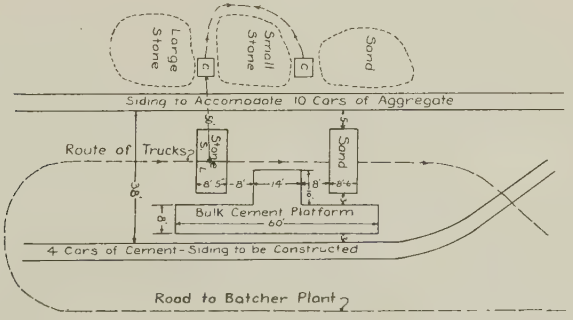


FIG. 15—TYPICAL STRAIGHT-LINE LOADING, ADAPTED FOR CEMENT IN BULK; TRUCKS LOADED MECHANICALLY OR WITH CONCRETE BUGGIES; SAND USED TO COVER CEMENT



FIG. 16—COST CHART FOR CONCRETE PAVING PROJECT 1

Cement hauled by truck directly to bulk cement plant. Aggregates from local producing plant—cost includes delivery to batching plant. Depreciation figured on 11.27 miles of pavement—life of paver taken as 55 miles. Concrete design: 5 sacks of cement per cu. yd. of concrete, 6 gallons water per sack of cement, and b/b$_o$ of .76. Mix by weight—1:2.67:4.67. Total cost per sq. yd. $1.362. Length of project 11.273 miles. Average length of haul 1.96 miles.

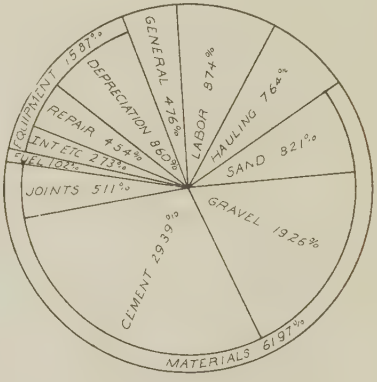


FIG. 17—COST CHART FOR CONCRETE PAVING PROJECT 2

Cement delivered in bulk by rail and handled in carts. Aggregates from commercial plant, delivered by rail—cost includes freight. Depreciation figured on 10.00 miles of pavement—life of paver taken as 50 miles. Concrete design: 5 sacks of cement per cu. yd. of concrete, 6 gallons water per sack of cement, and b/b$_o$ of .76. Mix by weight—1:2.71:4.68. Total cost per sq. yd. $1.472. Length of project 9.88 miles. Average length of haul 1.97 miles.

and finally dump the cement into the trucks.

On Wisconsin S. A. P. 2916, (Figs. 6, 8 and 12), the bulk cement was delivered by rail and the concrete buggies employed to handle the cement. On Wisconsin S. A. P. 2926 the cement was delivered on the job in trucks and the cement loaded mechanically. Both methods proved very satisfactory and the trucks could be loaded rapidly with no apparent loss of cement during loading or in transit. Sand was dumped on the coarse aggregate so that it could be pocketed and the bulk cement covered with the loose sand. Both jobs employed straight-line loading of the trucks with the bulk cement. This eliminated unnecessary turning or backing of the trucks.

Delaying a three-batch truck one minute represents a loss

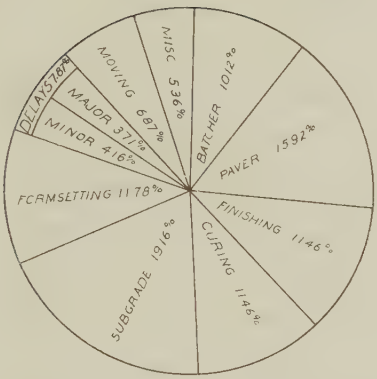


FIG. 18—LABOR COST CHART FOR CONCRETE PAVING PROJECT 1

Major delays: Unavoidable delays over 15 minutes in duration. Minor delays: Unavoidable delays under 15 minutes in duration. Cement handled by bulk cement plant and delivered directly by trucks. Moving includes that occurring during job. Concrete design: 5 sacks cement per cu. yd. of concrete, 6 gallons water per sack of cement, and b/b$_o$ of .76. Mix by weight—1:2.67:4.67. Total cost per sq. yd. $0.1260. Length of project 11.273 miles. Average length of haul 1.96 miles. Hauling by sub-contract—labor not included on chart.

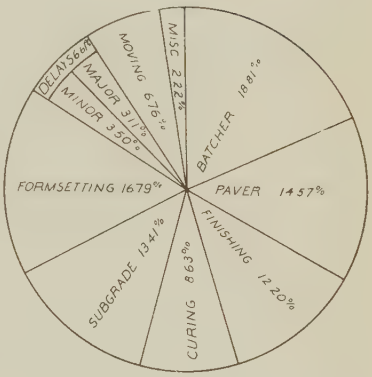


FIG. 19—LABOR COST CHART FOR CONCRETE PAVING PROJECT 2

Major delays: Unavoidable delays over 15 minutes in duration. Minor delays: Unavoidable delays under 15 minutes in duration. Cement handled in carts and delivered in bulk by rail. Moving includes that occurring during job. Concrete design: 5 sacks cement per cu. yd. of concrete, 6 gallons water per sack of cement, and b/b$_o$ of .76. Mix by weight—1:2.71:4.68. Total cost per sq. yd. $0.1287. Length of project 9.88 miles. Average length of haul 1.97 miles. Hauling by sub-contract—labor not included on chart.

of about five cents. This item may seem small but it accumulates rapidly into a larger sum. Delaying the mixer one minute often represents a loss of about one dollar. On a paving job where operations should synchronize and coordinate, seconds lost are dollars wasted. An efficient batching plant removes many chances for delay.

Tables 1 and 2 show summaries from the time studies made during June to October on the Sheboygan county, Wisconsin, jobs. These data were obtained by careful stop-watch studies made every day of all operations to determine the mixer delays in seconds of less than fifteen minutes in duration, which were classified as minor delays, and delays of greater than fifteen minutes, which were grouped as major delays. In both minor and

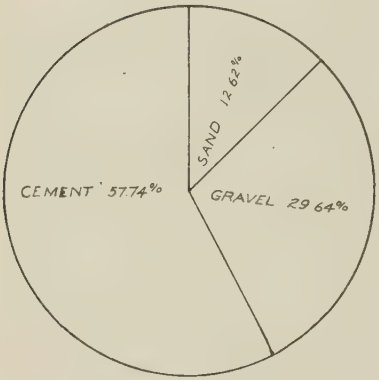


FIG. 20—COST CHART FOR CONCRETE MATERIALS PROJECT 1

Aggregates from local producing plant—cost includes delivery to batching plant. Bulk cement used. Concrete design: 5 sacks cement per cu. yd. of concrete, 6 gallons water per sack of cement, and b/b$_o$ of .76. Mix by weight—1:2.67:4.67. Total cost per sq. yd. $0.749. Length of project 11.273 miles. Average length of haul 1.96 miles.

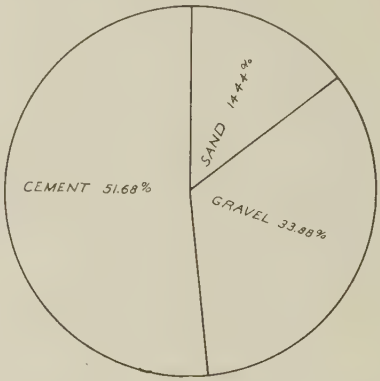


FIG. 21—COST CHART FOR CONCRETE MATERIALS PROJECT 2

Aggregates from commercial plant, delivered by rail—cost includes freight. Bulk cement used. Concrete design: 5 sacks cement per cu. yd. of concrete, 6 gallons water per sack of cement, and b/b$_o$ of .76. Mix by weight—1:2.71:4.68. Total cost per sq. yd. $0.837. Length of project 9.88 miles. Average length of haul 1.97 miles.

major delays, those believed to be unavoidable were classified as "A" and those believed to be avoidable as "B."

In noting the losses that occurred on the job, delays due to separated sizes are negligible, except on the job where the plant space was limited because of the four stock piles and the crowded conditions that existed. The job started with one crane but as production was improved. another crane had to be put on the job. Probable causes of delays were forecast on the job and through the contractor's cooperation in using suggestions to remedy them, delays of any consequence due to handling the separated aggregates did not occur.

Figs. 16-21 show the data collected on the Sheboygan county jobs. These charts illustrate the costs involved in constructing the concrete pavement, where both jobs had practically the same truck haul with the overall efficiency of one contractor 83.1 per cent, and the other 88.0 per cent. One job used local aggregate

TABLE 1—SUMMARY OF TIME LOSSES AND THEIR EFFECT ON PRODUCTION

Wisconsin S. A. P. 2, June 23 to August 21, 1930

Total *major* delays occurring during available working time	Class A		Class B			
Character	Hours	Per Cent	Hours	Per Cent		
Haul unit, supply			0.25	0.04		
Mixer trouble, mechanical	4.08	0.75				
Water supply	3.13	0.57	0.50	0.09		
Lack of materials at yard	0.77	0.14				
Crane trouble, mechanical	10.27	1.88				
Moving	83.45	15.30				
Rain	13.80	2.53				
Wet subgrade	21.25	3.90				
Miscellaneous	0.58	0.11	1.03	0.19		
					Hours	Per Cent
Totals	137.33	25.18	1.78	0.32	139.11	25.50
Time major equipment actually operated					406.07	74.50
Available working time, hours					545.18	100.00

Total *minor* delays occuring during time of actual operation	Class A		Class B			
Character	Hours	Per Cent	Hours	Per Cent		
Haul unit, supply			14.95	3.68		
Haul unit, operation	1.15	0.28	9.94	2.44		
Dumping	1.28	0.31	1.34	0.33		
Mixer trouble, mechanical	2.89	0.71	0.77	0.19		
Mixer operation	0.15	0.04	9.32	2.29		
Water supply			12.88	3.16		
Lack of materials at yard			0.33	0.08		
Subgrade not prepared			6.98	1.72		
Set parting strip			0.51	0.12		
Place reinforcing steel	1.27	0.32	0.07	0.02		
Wait for finishers	1.44	0.35	0.60	0.15		
Sand batch, lip curb	0.47	0.12	1.02	0.25		
Expansion joint			0.38	0.09		
Crane operation	3.03	0.74				
Miscellaneous	1.02	0.25	1.86	0.46		
					Hours	Per Cent
Totals	12.70	3.12	60.95	14.98	73.65	18.10
Time major equipment operated at 100 per cent efficiency					332.42	81.90
					406.07	100.00
Grand total of all Class B losses, hours					62.73	
Possible operating time, all Class B losses eliminated					395.15	
With all Class B losses eliminated, production would have been	18,972 batches				110,000 sq. yds.	
Actual production	15,945 batches				92,395 sq. yds.	
Overall efficiency of major equipment operation					83.1 per cent	

TABLE 2—SUMMARY OF TIME LOSSES AND THEIR EFFECT ON PRODUCTION

Wisconsin Project 1, Sheboygan County

Summary of all delays for job:

Total available working hours—613.0 Total production—116,347 sq. yds.

Total *major* delays occurring during available working hours	Class A		Class B		
Character	Hours	Per Cent	Hours	Per Cent	
Rain	31.75	5.19			
Wet grade	32.50	5.30			
Moving during job	35.25	5.75			
Material supply	24.75	4.04	1.50	0.24	
Subgrade	8.00	1.30			
Mixer trouble, mechanical	3.07	0.50			
Water supply	2.50	0.41	0.33	0.05	
Batcher	3.50	0.57	1.78	0.29	
Finishing machine	1.95	0.32			
Crane	5.50	0.90			
Joints			1.50	0.25	
Miscellaneous	4.75	0.78	1.10	0.18	
Totals	153.52	25.06	6.21	1.01	Total all *major* delays (Hours) 159.73 (Per Cent) 26.07
Time paver actually operated					453.27 73.93
					613.00 100.00

Total *minor* delays occurring during time of actual operation	Class A		Class B		
Character	Hours	Per Cent	Hours	Per Cent	
Hauling supply	0.19	0.04	7.46	1.65	
Hauling operation	0.11	0.02	4.20	0.93	
Dumping	0.84	0.19	5.41	1.19	
Mixer trouble, mechanical	4.81	1.06	0.09	0.02	
Mixer operator	0.28	0.06	8.18	1.80	
Water supply	4.94	1.09	6.36	1.40	
Joints and steel	2.63	0.58	2.30	0.51	
Subgrade	0.28	0.06	3.32	0.73	
Miscellaneous	7.37	1.63	5.42	1.19	
Totals	21.45	4.73	42.74	9.42	Total all *minor* delays (Hours) 64.19 (Per Cent) 14.15
Time paver operated at 100 per cent efficiency					389.08 85.85
					453.27 100.00

Grand total all Class B time losses, hours 48.95
Possible operating time all Class B losses eliminated, hours 438.03
Operating efficiency of paver 99.1 per cent
Time efficiency of paver 88.8 per cent
Overall efficiency of paver 88.0 per cent
Total possible production at 100 per cent overall efficiency—132,267 sq. yds.

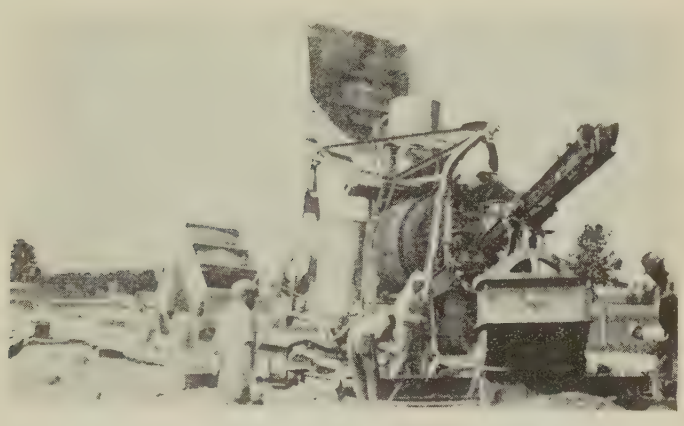






FIG. 22—TYPE OF 1926 MODEL—27-E PAVER USED ON S. A. P. 2916 AND S. A. P. 2926, SHEBOYGAN COUNTY, WISCONSIN

FIG. 23—TYPES OF 1930 MODEL—27-E PAVER USED ON S. A. P. 2916 AND S. A. P. 2926, SHEBOYGAN COUNTY, WISCONSIN





FIG. 24—METAL SAMPLE BUCKETS AND BEAM MOLDS IN POSITION TO RECEIVE CONCRETE AS DUMPED FROM MIXER SPREADER BUCKET

FIG. 25—REMOVING SAMPLE BUCKETS AND BEAMS FROM THE SUBGRADE TO THE BERM OF THE ROAD WHERE ANALYSIS IS MADE ON THE FIVE PARTS OF THE BATCH FOR THE EXACT AMOUNT OF GRAVEL, SAND, CEMENT AND WATER

and the other commercial aggregate; five sacks of cement per cu. yd. on each job.

Previous to 1930 approximately six sacks of cement were used per cubic yard of concrete on most State and Federal paving jobs. By selective grading and reducing the void content of the coarse aggregate on these two jobs, the cement reduction to five sacks per cu. yd. was made without loss of strength. A saving of approximately 6.5 cents per sq. yd. was made. Even with this reduction of cement content, the cement still forms about 30 per cent of the total cost of the pavement, and on the



FIG. 26—WASHING THE CEMENT OUT OF THE SAMPLES OF CONCRETE THROUGH A NEST OF NO. 4, 48 AND 100-MESH SIEVES

basis of the cost of the materials alone—that is, the gravel, sand and cement—the percentage is over 50 per cent. (Investigations are in progress to study the effect of cement content on the durability of concrete.)

Further reductions in hauling, labor and depreciation will be made next year in Wisconsin by the use of a 33-ft. batch and a 50-sec. mixing time after all solid material are in the drum.

On these two Sheboygan county jobs and on a Federal-aid project in Arkansas, 242 batches were sampled as they were dumped on the subgrade and tests made to determine the exact amount of coarse aggregate, sand, cement and water in samples taken from the four corners and center of the batch. From these same batches, beams and cylinders were also made—a total of 1,276 cylinders and 726 beams. Cylinders were broken at 28 days and the beams once at 7 days and twice at 28 days. The curing was the same on all jobs. The gravel obtained from the concrete sampled in the field was sieved.

These data were all obtained in connection with our study of batch size and mixing time. Table 3 gives the grand total of all data collected on each job.

The Sheboygan jobs show a slightly higher cement content than five sacks per cu. yd., because less water was used than anticipated. The water content was supposed to have been six gal. per sack of cement.





FIG. 27 (LEFT)—WEIGHING EACH BATCH PART IN AIR AND THEN IN WATER, USING THE APPARATUS SHOWN. WEIGHING WAS ACCURATE TO $\frac{1}{16}$ OUNCE

FIG. 28 (RIGHT)

One cylinder was obtained from the four corners of the batch and two from the center. Three 6 x 8 x 42-in. beams were taken from each batch. Cylinders made according to the A. S. T. M. methods. Beams were only rodded at the edges without disturbing the center mass of concrete.

On the Arkansas job the cement was supposed to have been 5.40 sacks per cu. yd. of concrete and 5.25 gal. of water per sack of cement. Because of the high temperature and high evaporation, more water was used.

The coarse aggregate used in Arkansas was a crushed trap rock and in Sheboygan, Wisconsin, a limestone gravel. The maximum size of aggregate on both jobs approximated 2½ in.

The variation on all the jobs approximate each other within one per cent and are fairly comparable in regard to actual cement and water content. Please note that the variations from batch to batch were less than the variations within the batch.

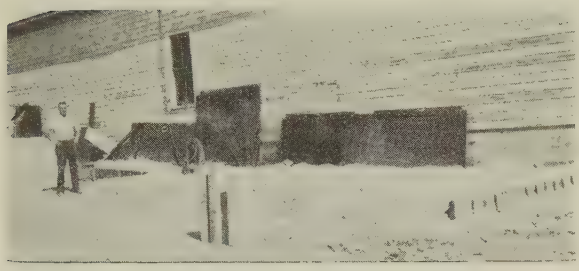


FIG. 29

Beam specimens were cured in damp sand for 28 days and tested in the field. Cylinders were cured at the same location as beams and removed in time to send to the Madison, Wisconsin, State Forestry Laboratories to be tested at 28 days.

TABLE 3—FINAL SUMMARY FOR JOBS

Job No.	No. of Batches Sampled	Proportions in Percentage by Weight				Per Cent Variation within Batch	Per Cent Variation Batch to Batch	Sacks Cement per cu. yd.	Water per Sack	Ratios by Absolute Volume			Mix by Weight	
		Gravel	Sand	Cement	Water					a/c	b/a	b/bo	Sand	Gravel
*1	118	52.56	30.11	11.35	5.97	3.90	3.07	5.16	5.96	3.02	1.72	0.746	2.67	4.67
*2	80	52.11	30.34	11.28	5.73	4.37	3.22	5.18	5.81	3.03	1.72	0.750	2.71	4.68
†4	44	48.56	33.40	11.78	6.01	4.75	3.35	5.19	5.82	3.38	1.45	0.751	2.86	4.18

*Wisconsin
†Arkansas

Job No.	Cylinders		Beam Strength						Sieve Analysis Percentage retained on				
	[1]Cylinder Strength	[2]Per Cent Variation	7 Day	[2]Variation	End[1]	[2]Variation	Center[1]	[2]Variation	2½"	1½"	¾"	⅜"	4"
1	3646	6.33	598	4.55	872	4.03	866	4.76	4.0	14.8	44.3		36.9
2	3672	5.66	595	4.70	877	5.22	868	4.95	3.6	13.6	38.1		44.6
4	5039	5.89	675	6.56	723	6.04	698	5.48		21.1	56.3	16.3	6.3

[1]Breaking strength in pounds per square inch at 28 days.
[2]Variation of cylinder strengths within batch

Job No.	Maximum Cement	Minimum Cement	Maximum Water	Minimum Water	Maximum Cylinder Strength	Minimum Cylinder Strength	Beam Strengths					
							7-Day		End[1]		Center[1]	
							Max.	Min.	Max.	Min.	Max.	Min.
1	5.49	4.83	6.38	5.58	4031	3266	633	562	919	826	919	814
2	5.64	4.69	6.55	5.15	4012	3325	631	560	934	819	918	810
4	5.72	4.63	6.69	5.12	5323	4756	737	621	779	673	748	648

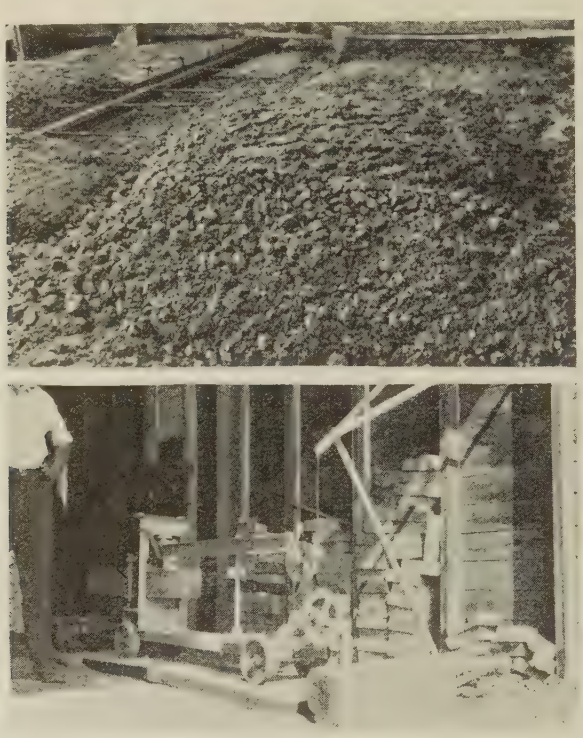


FIG. 30

The last cubic foot of concrete discharged from any make of mixer has a predominance of coarse aggregate. Therefore approximately a cubic foot of concrete should be retained in the mixer drum. Also the design of mixer spreader bucket causes segregation. The results of these two cause a batch to appear such as shown in the above photograph.

FIG. 31—THE BEAM TESTING MACHINE AND THE LOAD OF APPLICATION WAS CONTROLLED TO BE THE SAME ON ALL SPECIMENS AND TESTED BY THE SAME OPERATOR

Even by visual inspection one could see on these jobs that variations from batch to batch were reduced by using separated sizes for the coarse aggregates. This was valuable in reducing delays at the mixer, where ordinarily a concrete produced with a very low water content and low cement factor gives occasionally an extremely dry batch which almost invariably delays the finishing operations.

Variations within a batch will apparently be hard to eliminate

no matter what the size of the batch (our tests cover batches varying from 27 to 35 cu. ft. in a 27-E paver), or whether the concrete is mixed more than 50 seconds after all the solid materials are in the drum. The variations within a batch become larger as the range from small size aggregate to large size aggregate becomes larger.

In general terms a slab is no stronger than its weakest batch, and each batch is no stronger than its weakest part. Therefore, we must work toward the most advantageous equalization of both the variations between batches and the variations within the batch. It follows we should not attempt to have the variation from batch to batch reduced by too costly refinements below the variation within a batch. This, I believe, is the answer to the question, "How many separated sizes of coarse aggregate should be used?"

To reduce variations from batch to batch beyond a certain point costs money. In some, probably many cases, the use of separated sizes of coarse aggregate can be made to yield a net profit. The extent to which this process can be pushed, however, seems to have a definite limit, in that the cost is likely to increase progressively as the variations are reduced. To press the reduction in the variations from batch to batch too near the same value as the variations within the batch would therefore bring us within the field of diminishing returns. And to bring the variations between batches below the variations existing within the batch would seem to be of little use, even though this could, by some happy chance, be accomplished at no extra cost.

If oversanded mixes are still retained after the introduction of the use of aggregate of multiple sizes, the added expense is a loss. It is obvious that by using separated sizes intelligently a lower mortar ratio can generally be used, and as a consequence the cement factor can be reduced. The extent to which this can be done for any given extra cost will form a measure of what saving can be expected from using separated sizes of coarse aggregate in concrete road construction.

DISCUSSION

BY A. L. HAMBRECHT*

The Department of Commerce reports show one hundred million barrels of cement manufactured in 1920 which represents an increase of approximately 26 per cent over the amount manufactured in 1910 and a further increase of approximately 70 per cent from 1920 to 1930. A large part of the increase shown for the 1920-1930 decade is accounted for by the increased highway construction programs of that period.

To meet the demands of the added construction there followed a development and improvement of various types of paving equipment which is probably unparalleled in any other line of industry. Compare the modern concrete paving outfit of comparatively few men and nearly perfect coordination between the various units with the more or less disjointed, inefficient paving outfit commonly used prior to 1920—a large number of men who spent a considerable part of their time side-stepping to avoid colliding with each other—wooden forms in various stages of disintegration—stock piles of aggregates on the subgrade, more likely than not covered with a liberal layer of dust—a maze of plank runways—a collection of decrepit wheelbarrows—a wheezy old alley steam paver—and last but not least the man-killing, hand operated strikeoff boards. As we contemplate this we cannot help but wonder why the old order held forth as long as it did.

The sudden and unprecedented increase in the use of portland cement concrete for paving caught highway engineers more or less unprepared with the result that, except in a few isolated cases, the art of concrete manufacturing from the standpoint of the concrete itself did not receive a great deal of attention. Arbitrary proportions were specified—the aggregates were proportioned by volume without correction for bulking—the amount of mixing water added to the batch was left entirely to the judgment of the mixer operator with the result that the batch to batch consistency varied greatly.

The importance of controlling and maintaining uniform amounts of aggregates in the batches and maintaining a uniform con-

*Construction Engineer, Wisconsin State Highway Commission.

sistency was recognized and resulted in the almost universal adoption of weight proportioning and the installation of accurate water measuring devices on paving mixers.

The several theories of proportioning and designing concrete mixtures whether it be by arbitrary assignment, fineness modulus, maximum density, water cement ratio or voids recognize the importance of maintaining uniform proportions of the materials if uniform results are to be obtained. This necessitates accurate measurement of all materials. The character of sand is such that segregation in handling is negligible and the volume of the given sand is approximately uniform when dry. Sand can easily be weighed to a high degree of accuracy and moisture corrections easily and accurately made. The coarse aggregate, however, presents a more difficult problem. By weighing a definite absolute volume can be maintained. The apparent volume, however, will vary as the gradation varies. When we consider that we are dealing with a material with a range in gradation from 2½ in. to ¼ in. or 1½ in. to ¼ in., and the frequent handling required between the producing plant and the placing in the forms it becomes apparent that wide variations can and do exist from the gradations originally specified. Even with the utmost care in stock piling we find an accumulation of finer particles in the center and an excess of coarser particles on the outside of the pile.

The tendency of coarse aggregate to segregate and the resulting variation in apparent volume, voids and surface areas, which in turn affect uniformity of water content, mortar content, workability and yield is a real obstacle to concrete control. Variations of 15 per cent in the voids of equally proportioned coarse aggregates due to segregation and poor gradation frequently occur and such variations can easily cause a 10 per cent batch to batch variation in cement content per unit volume of concrete.

The separation of coarse aggregate into several sizes, thereby maintaining the voids at a uniform minimum and the use of a uniform minimum amount of mortar consistent with workability will tend to insure a reasonably uniform product for any given combination of materials.

During the 1930 construction season the use of coarse aggregates in separated sizes was required by the Wisconsin Highway Commission on a number of paving and bridge projects. The

materials used consisted of those most commonly used in concrete work in this state, namely, calcareous or dolomitic gravel, igneous or granitic gravel and crushed limestone. The aggregate for one project was produced from a local pit and the other projects were supplied by commercial plants.

The coarse aggregate for the paving projects was required to be furnished within the following gradations: limestone and calcareous type gravel in three sizes, ¼ in. to ¾ in., ¾ in. to 1½ in. and 1½ in. to 2½ in. Igneous type gravel in two sizes, ¼ in. to ¾ in. and ¾ in. to 1½ in. For bridge work the maximum size allowed is 1½ in.

The production of coarse aggregate in separated sizes whether by a commercial plant or by the contractor from a local pit will in most cases require plant changes consisting principally in providing increased screen capacity to permit a close separation of sizes. Commercial producers will readily agree to accept this method as they have for some time been attempting to effect a standardization of commercial sizes.

On our 1930 work, which can be considered of an experimental nature, no difficulty was experienced in securing the required sizes from commercial plants or in the local pit production.

In handling rail-shipped aggregates from cars to stock piles or proportioning bins, side track and yard space available will control the size of stock piles and general arrangement of plant and equipment. Two double compartment bins will be required when using three sizes of coarse aggregate and two cranes will be necessary to avoid delays except under the most favorable arrangements. The pictures shown by Mr. Thee give some idea of the restricted space available for stock piles, proportioning bins and cement handling platform on the Johnson job. The arrangement while inconvenient did not appreciably affect production.

On the project where two sizes were used a single crane easily kept the three compartment bin fully charged.

With the locally produced material the aggregates were hauled by truck from the producing plant and stock piled on an area adjacent to the project. The arrangement of bins and stock piles enabled a single crane to handle the material without causing any delays.

The use of two bins introduces added costs for equipment and the extra batch operator and involves some lost time for the trucks. The additional batching time required will depend upon the distance between and the accessibility of the bins. With two sizes of coarse aggregate, using a three compartment bin there was no noticeable increase in the time required for batching.

With all of the materials entering each batch accurately controlled the resulting concrete as deposited on the subgrade has a decidedly uniform appearance. There were no batches with an excess of coarse aggregate or an excess of mortar, a common occurrence on projects using a single size of coarse aggregate. Other factors being equal, uniformity of concrete will to a great extent eliminate the unequal shrinkage or settlement responsible for many of the present pavement surface irregularities.

The views showing the placing and finishing of the concrete indicate an excess of mixing water. This did not generally prevail on our work and we try to maintain a consistency which will give a slump of about 1½ in. and further limit the mixing water to 6 gal. maximum per sack of cement.

A comparison of results obtained on two projects, one using separated sizes and the other a single size, indicates that close control of the proportioning of aggregates produces concrete of greater uniformity. As the following will indicate, the projects were built at the same time of the year and were within 30 miles of each other. The aggregates were supplied from the same commercial plant and the proportions were identical except for the separated sizes. The average strength of the specimens using separated sizes was 11 per cent greater than the average strength using the single size. On the job using recombined aggregates all breaks were within 10 per cent above or below the average and of the total of 53 breaks only two tested more than 5 per cent above and only 6 tested more than 5 per cent below the average.

The variation in strengths on the job using the single size of stone ranged from 25 per cent above to 25 per cent below the average and of the total of 41 breaks 16 tested more than 5 per cent above and 16 tested more than 5 per cent below the average.

These results are generally typical of the results obtained on other projects.

An analysis of the concrete as it is discharged from the spreader

bucket indicates that there is a variation in the relative proportions of coarse aggregate, fine aggregate, cement and water in different parts of the batch. In a typical series of tests the following variations existed. The cement content varied from 4.79 sacks to 5.42 sacks per cubic yard. The water varied from 4.96 gal. to 5.45 gal. per sack of cement. The sand cement ratio varied from 2.99 to 3.14. The gravel sand ratio varied from 2.10 to 1.53 and the ratio of the absolute volume of coarse aggregate in a cubic foot of concrete to the absolute volume of coarse aggregate in a cu. ft. of coarse aggregate or the b/b_o varied from .845 to .749.

It is therefore evident that unnecessary refinements in gradation which would tend to increase costs would be unwarranted unless the interior of the mixer drum and spreader bucket can be designed to overcome the variations noted above.

The results obtained by the use of separated sizes of coarse aggregate on the 1930 work have convinced us that it is a construction detail that should be recognized in designing concrete mixes, when a relatively high percentage of coarse aggregate is used. The type and character of materials available and the combinations possible with them can materially affect the final cost without sacrificing quality.

BY GEO. LANGLEY, JR.*

My position as County Highway Engineer placing me, as it does, in direct field contact with construction work, and in direct contact with the people to whom I am responsible (the tax-paying public) necessarily inclines me to consider the theories involved in the design of concrete mixtures, from the thoroughly materialistic rather than from the profoundly abstract viewpoint.

From 1927 to 1930 inclusive, Sheboygan county has constructed 80 miles of portland cement concrete highway pavement.

In 1927, 5 miles were constructed using the Wisconsin Highway Commission specification, 1:2:4 proportioning with all measurements by volume.

Twelve miles were constructed in 1928 again using the Wisconsin Highway Commission specification 1:2:4 proportioning, the coarse aggregate measured by volume, and the fine aggregate

*County Engineer, Sheboygan County.

measured by weight with corrections for moisture content and bulking factor.

Thirty miles were constructed in 1929 using a designed mix, both aggregates being measured by weight, without separating and recombining the coarse aggregates.

Thirty-three miles were constructed in 1930 using a designed mix, measurements by weight and the coarse aggregate separated into three sizes and recombined in the batch.

Of these 33 miles about 23 miles were constructed under the observation of, and subject to tests by the Division of Management, U. S. Bureau of Public Roads. Specifications on this 23 mile construction called for the use of four different sized batches, 27, 30, 33 and 35 cu. ft. as directed by the engineer in charge. Unit prices were received on each batch size, the average contract price of the 27 cu. ft. being 3¢ per square yard greater than the 30 cu. ft. batch, and the 30 cu. ft. being 2½¢ more than the 33 or 35 cu. ft. batch.

Test results on the concrete produced during these four years were as follows:

Year	Flexural Strength		Compressive Strength		
	7 Day Beams Average Mod. of Rup. lbs. per sq. in.	28 Day Beams Average Mod. of Rup. lbs. per sq. in.	Cylinders 28 Days lbs. per sq. in.	Cores-Age 6 Months lbs. per sq. in.	Cement Content per cu. yd.
1927	None	None	None	None	1.6 Bbls.
1928	None	None	None	3960	1.39 Bbls.
1929	521	665	None	4250	1.24 Bbls.
1930	590	861	3665	To Date 5680	1.25 Bbls.

The tests taken and records kept during 1930 by the Division of Management, U. S. Bureau of Public Roads were very complete. Sheboygan county has a definite program calling for the construction of 45 miles of the same type of highway. It follows, of course, that we, desiring to profit by the records of past experience, are making an exhaustive study of these records.

The outstanding facts gleaned from our test records of the past four years are:

1st: An increase of 40 per cent in compressive strength in 1930 as compared to 1928.
2nd: An increase of 30 per cent in flexural strength in 1930 as compared to 1929.
3rd: Constantly increasing control of proportioning of aggregates.
4th: Constant decrease in the quantity of cement per cubic yard of concrete.
5th: Constant increase in rate of production.

Certainly, the increased strength of the concrete, together with an increase in density, should indicate increased durability, at least insofar as the effects of changes in temperature are concerned.

The decrease in cement content made possible by careful proportioning of the aggregates and the increase in production due to increase in batch volume and decrease in mixing time are, of course, of enormous economic value.

It will be noted by reference to Fig. 3 and 4 of Mr. Thee's paper that by decreasing the cement content from six to five bags per cu. yd. there was a reduction of 6½¢ per sq. yd. in the cost of the pavement. As Sheboygan county built 33 miles of concrete pavement last year, the saving in cement cost was $26,100.00.

The saving in cost due to the increase in the volume of the batch amounted to 5½¢ per sq. yd. Consequently, the saving due to this item was $21,240.00, or a total saving on the 33 miles of $47,340.00. This total of $47,340.00 was 20 per cent of the total general property tax for highway purposes in Sheboygan County in 1930.

It is evident from the records, that close control of the aggregates makes possible a considerable saving in cost.

In our experimental use of three sizes of coarse aggregate, we confined ourselves to the use of such equipment for this separation as was available and made every effort to avoid changes in this equipment.

Coarse aggregate proportions to fit the existent production conditions were used.

Screen equipment at production plants was such as to make it impossible to obtain efficient separation. An over run of sizes sometimes as great as 50 per cent was noted.

It is evident that if crushers are adjusted to produce coarse

aggregate in the most advantageous proportions and sufficient screen area used to insure complete separation, much closer control of the proportioning of the aggregates may be obtained.

Records from other localities show that coarse aggregate with a maximum size of three and one-half inches is being used with no detrimental effect on the strength of the concrete. The use of a 3½-in. maximum size of coarse aggregate would reduce the cost of production of the aggregate. It would also make it possible to obtain a larger percentage of absolute volume in a cubic yard of coarse aggregate.

Bins are already planned by equipment manufacturers to make it possible to load the truck with one stop, thus eliminating much of the extra truck cost of last season.

Inasmuch as it has been our experience to note, and our records plainly indicate, that as we gained closer control of the proportioning in the materials, the quality of the concrete improved and the cost decreased. I find it hard to accept Mr. Thee's conclusion that two sizes of coarse aggregate are sufficient.

Mr. Thee's conclusion is based, of course, on cost records of last year, and I fear that the methods used in the separating and recombining of the coarse aggregate were too crude to reflect true cost.

It seems possible that the continued use of three sizes of coarse aggregate may bring about more efficient equipment and methods, so that the benefits derived will warrant the cost.

I believe that with control of proportioning as near perfect as possible, the cement content per cu. yd. of concrete need not be arbitrarily specified, but that it is dependent on the gradation of the materials used.

It seems evident that if in a cubic yard of concrete, the greatest amount of absolute volume of coarse aggregate practical, from the viewpoint of workability, is used, the amount of mortar required will be the minimum.

It seems just as evident, that if the absolute volume in the quantity of sand required for the mortar is the greatest obtainable, the cement content requirement of the mortar will be the minimum.

Such a combination of materials should not require too large an amount of water to bring about the necessary degree of work-

ability, and even if too much water is placed in the mixture, the excess quantity will be eliminated in placing and finishing the slab, so that a proper water cement ratio will be effective before hardening takes place. It is our belief that the water cement ratio of concrete in a highway slab pavement is in no way affected by the quantity of water, within reasonable limits, placed in the mixer, provided, of course, that there is sufficient water to hydrate the cement used; also, that the quantity of water, after the concrete is in place on the subgrade and has received its final working, and is ready to set, establishes the effective water cement ratio, and that this ratio is to a considerable extent dependent upon the gradation of the dry materials used in the concrete.

If materials of a proper gradation are obtainable, a computation of proportions is both possible and plausible, and this computation is a matter of field practice and does not necessarily need laboratory test.

To me the test records of our work of last season indicate that the cement requirement per cu. yd. with which to obtain the greatest strength possible can be computed, and that it is materially (at least 10 per cent) less than that used last season.

After our experimental experiences of the past in the design of concrete mixtures, I feel that tests should be taken in the field to determine the relative values of the various different types and brands of cement.

In conclusion, I wish to express my appreciation of the fact that I was allowed a small part in the cooperative studies of last year, and the hope and desire that these cooperative studies be continued during the coming season. Surely the benefits derived from field research are worth the investment.

Readers are referred to the JOURNAL *for December 1931, (Vol. 28) for further discussion which may develop. Such discussion should reach the Secretary by Nov. 1, 1931.*

Design and Operation of Central Mixing Plants

*Progress Report of Committee 603**

BY FRANK I. GINSBERG† AUTHOR-CHAIRMAN

The development of the central concrete plant as a commercial undertaking has been so rapid that there is yet scarcely time in which to have built up any "best designs" or "best operating methods" which are complete and comprehensive. It seems the proper function of the committee now, as Mr. Ginsberg has written, to outline recent progress, note some clearly developed pitfalls to be avoided and to stimulate discussion among the many active in plant design and operation. Several members of the committee participated in a symposum at the 26th Annual Convention, New Orleans, reported in this Journal March 1930, Vol. 26, p. 551. The present report prepared by the author-chairman has been submitted to the critic members of the Committee: Joseph E. Burke, Miles N. Clair, N. D. Crowley, G. L. Lucas, A. W. Munsell, H. F. Thomson, Fred C. Wilcox, all of whom give it general approval and some of whom expect to submit independent discussion. Mr. Ginsberg acknowledges his gratitude to the members of the committee for their general cooperation.—Editor

INTRODUCTORY

In the last three years there have been marked changes in the design of central plants due to the development of the ready-mixed concrete business, which has rapidly changed the picture in the manufacture of concrete. The primary purpose of this report is to outline progress in central plant design during this period, and second to develop discussion that will result in a

*Presented at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.
†Vice-President, H. O. Penn Mach. Co., Inc., 140 St. and East River, New York City.

sound basis of control and the most economical production methods in the manufacture and delivery of central plant concrete. It is a far cry from the old haphazard methods of mixing concrete by the feeding of materials measured and controlled by the rule-of-thumb method, to the present precise mechanically controlled weighing devices and efficient material handling machinery. Much has been accomplished in this direction and the future developments in this industry will bring much more efficient methods in the control and the economic handling of central mixed concrete.

The conditions that will govern the design and arrangement of a plant for the production of ready-mixed concrete are controlled by a number of local factors. The location of the plant with reference to the possible consumption and use of concrete is important. The size and type of plant will of necessity be entirely controlled by these local conditions.

It is surprising to note that in towns with a population of 15,000 to 20,000 there have been as many as three plants in operation, all of which seem to be getting along successfully. This, however, is most unusual and is not generally practicable. There is, however, a potential outlet for ready-mixed concrete in small localities where there is considerable road work or street paving to be done, which may in some cases justify larger plants than really indicated by the more obvious conditions.

The method of receiving materials, whether by rail, truck or water, also controls the size and type of plant.

There has been considerable difference of opinion, whether the most efficient and practicable plant is to be the central mixing plant or the central proportioning plant.

TYPES OF CENTRAL PLANTS

The central mixing plant is one in which the materials are proportioned and mixed in a concrete mixer at the plant and delivered in agitating or non-agitating trucks to the point of final deposit.

At a central proportioning plant materials are all proportioned and batched into a mixer type of truck body, in which mixing is done enroute with water added to the batch in the truck, or mixed on arrival at the job.

Each type of plant has some advantages and from such tests as have been made at the time of leaving plant and the time of final deposit, there seems to be no appreciable difference.

There is also a design of plant recently developed which combines a central mixing plant and a central proportioning plant, and which is used for both purposes as occasion requires.

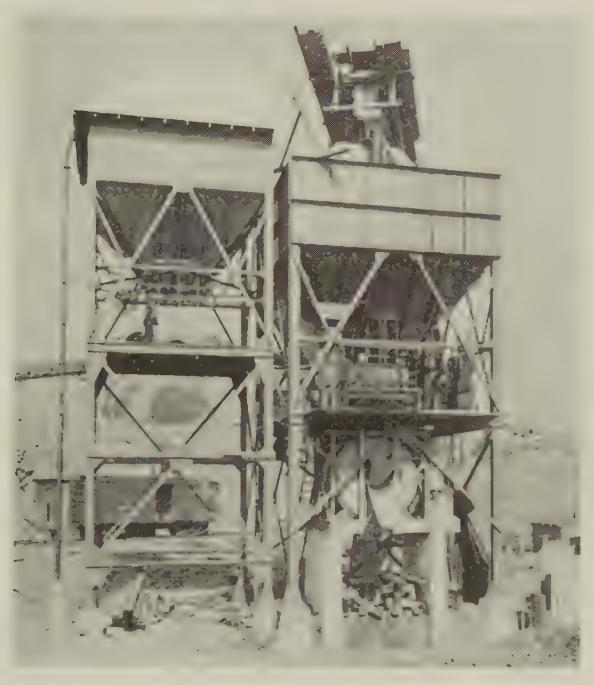


FIG. 1.—LOIZEAUX BUILDERS SUPPLY CO., ELIZABETH, N. Y.

PLANT EQUIPMENT AND METHODS

Bins. Steel bins have come into general use in nearly all types of central plants, and the weighing method of controlling the proportions of materials has supplanted the older methods of volumetric measurement. Material bins are generally divided for the storage of from four to six classes of aggregates and there are some occasions where storage of eight classes of materials has been provided for, but this is not usual.

Unloading Facilities. Where water facilities for receiving materials are available, the material is generally unloaded by crane or derrick directly into the bin. In some cases derricks are preferred to cranes because of the height of the bin, which ranges from 35 to 50 ft., depending of course, on the size and capacity of the bin itself. In under favorable conditions a crane unloads into a small hopper at the dock, and belt conveyors elevate the materials to storage bins. Cranes are faster in unloading where bin heights permit their use.

Conveyors deliver materials to a revolving spout on the top of the bin, which diverts the flow of materials into the proper compartment. The revolving spout is generally controlled from the platform, where the operator governs the proportions of the materials.

Where materials are delivered to the plant by rail the general method is as follows: Bottom-dump standard gauge cars drop the materials into a track hopper, from which reciprocating feeder or belt feeder delivers to a belt conveyor, thence to the top of the bin, and in some cases to a bucket elevator, or where conditions permit, the materials are delivered directly to a bucket elevator from the feeder, which in turn elevates the material to the top of the bin into a revolving type of hopper, from an extended chute at the top of the elevator. The unloading of standard gondola cars by derrick or crane clamshell bucket methods is also used extensively.

The vertical type of steel-enclosed elevator seems to have worked out best on permanent installations of this type. The choice of elevators is most important and the proper design of this unit and capacities must be given careful consideration. It is recommended that in large plants elevators have a minimum capacity of 100 cu. yds. per hour and be preferably, of the continuous type with double-strand chain and steel buckets; and that automatic brakes be installed against possible power line troubles while this unit is in operation.

Where the delivery of the materials must be made to the plant by truck, the general practice is: To arrange a form of grill work where the truck discharges the material directly into the boot of the elevator or into the hopper of a belt conveyor. There should be a gate on the hopper of the belt conveyor or bucket elevator

to permit control of the feeding of the materials on the respective units. In some instances, a long belt conveyor is used and the material discharged onto the belt conveyor through a series of gates. This is known as the tunnel belt method of storing and handling aggregates and is in successful use in a number of plants throughout the country. This belt runs horizontally under the stock piles and is then inclined to the revolving hopper on top of bin. The angle of this inclined section is generally from 18 to 20 deg. and in no case should this angle exceed 24 deg.

A method of delivering concrete materials in containers transported on gondola cars has been developed recently by some of the eastern railroads. The containers are open-top rectangular steel boxes, usual capacity about six tons, with twelve boxes on each car. The containers are of bottom-dump type; are handled by crane or derrick and can be used for sand, stone or cement.

HANDLING CEMENT IN BULK

The use of bulk cement is developing very rapidly, and suitable provision is being made by the manufacturers of bin and batching equipment so that the control of this material is properly coordinated with the control of the fine and coarse aggregates. There are a number of methods for handling bulk cement.*

One method is in delivery of cement in box cars or specially designed bottom-discharge cars from which cement is unloaded into hoppers through screw conveyors and elevated into cement storage bins. Cement from the storage bin is then handled by various devices such as screw conveyors, bucket elevators, into auxiliary storage bins directly over the cement weighing hoppers or by screw conveyors with electric push button controls feeding directly into a cement weighing hopper. It is suggested that in all cases where cement is being used in bulk, that it be delivered into a separate compartment for cement only after being weighed and then discharged into the mixer through properly arranged chutes.

There have been a number of instances of cement being weighed in the same hopper in which the aggregates have been weighed. This, however, is not recommended because the cement will build up on the wet surfaces of the aggregate weighing

*Committee 607, Use of Cement in Bulk. Herbert Coffman, author-chairman, presented a report at the 27th Annual Convention. It is scheduled for publication in the JOURNAL for Sept. 1931, Vol. 28.—EDITOR

hopper. The use of compressed air equipment for unloading and delivering cement directly into the cement storage bin is coming into more use and devices have been developed for unloading the cement directly from box cars or boats and pumping it into the storage bins without the additional use of power shovels or hand shovel methods that have been necessary in handling bulk cement to date.

The compressed air pumping system is widely and successfully used in the handling of bulk cement in central plants. Electric power is generally used for the cement pump and for the compressor unit. Automatic signal arrangements have been devised to show when the bins are full, to prevent overloading. It is well to have a supply of compressed air where bulk cement is being used, regardless of the method of delivery into the bins. This air supply is used for fluffing up the cement after the cement in the bins has settled and packed. The introduction of a small amount of air into the bin or container has the effect of causing the cement to flow more easily through the various valves and gates and eliminates much of the arching action over the openings of the bins and containers. Care must be taken to keep the air free from excess moisture. The use of specially designed cement gates is strongly recommended properly to control the flow of cement and also to prevent loss of cement through leakage.

Bins for storage of bulk cement should be properly designed and well constructed and made water-proof and leak-proof. The use of lead strips and lead gaskets in all bolted and riveted steel connections of the bin proper is recommended. The angle of the bottom slope should not be less than 55 deg. Roof may be of wood or steel and must be made tight and with necessary openings for easy loading in accordance with the method of delivering cement to the top of bin.

Bucket elevators and screw conveyors where used, should be dust-proof and water-proof, and with properly designed openings for cleaning; and for the adjustment of bearings and for replacements when necessary. Electric motors should be properly enclosed and protected against the elements and made dust-proof as far as possible.

Provision for the storage of more than one kind of cement should be given serious consideration. Two or three compartments are generally desirable with the proper arrangement of gates and diversion spouts for delivery to the weighing hoppers. Note: Bag cement is also successfully used in conjunction with bulk cement or separately where conditions warrant.

CENTRAL MIXING PLANTS—AGITATOR OR NON-AGITATING TRUCK DELIVERY

Proportioning methods in general use for the operation of wet plants, are practically the same as in central proportioning plants designed for the operation of mixing in transit. The weighing batchers should be at least 50 per cent larger than for the maximum batch to be weighed. There are a number of methods of measuring by weight.

1. The use of a one, two or three beam scale with electrically controlled quick reading weighing device.

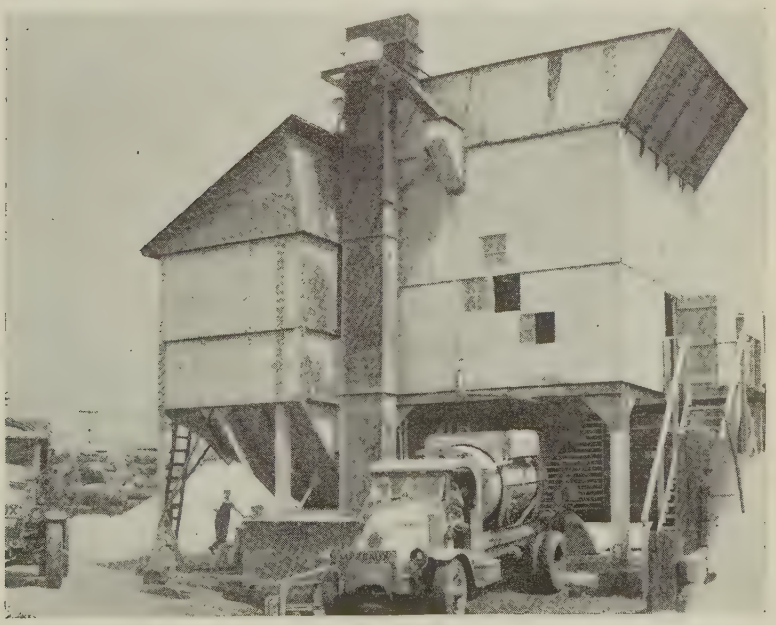


FIG. 2.—READY MIX AND SUPPLY CORP., ALBANY, N. Y.

2. One, two or three beam scales with indicator colored dial for quick reading purposes.

3. Large dial scale for weighing cumulative quantities, the various proportions being shown by indicators on the rim of the dial itself, and the total weight of all the materials representing the quantity in the batcher. The gates controlling the flow of the material into the weighing hopper are of two kinds—the sliding gate and the clamshell gate. The sliding gate is generally controlled by a rack and pinion arrangement with wheel controls, and the clamshell or radial gates are controlled with levers. It is important that the levers or wheel controls all be so arranged that the operator can control the various materials from one point on his platform. It is recommended cement not be weighed on aggregate material scales.

The tripping device for discharging the materials into the mixer are either of the direct trip or gate controlled discharge type.

The angle of the sides of the weighing hopper as well as the angle of the aggregate storage bin itself, should be of sufficient pitch so that the hoppers will be self-cleaning and not permit the arching of materials, which would necessitate pounding or hammering to accelerate the flow of material into the mixer. A 50 deg. angle is the flattest slope that is practical.

It is desirable that the size of the batchers be large enough to deliver a full batch to the size of the mixer used. The writer has observed in some plants the use of one yard batchers where a two or three yard mixer is being used. This necessitates two or three separate operations to measure one batch of concrete, with loss of time and economic inefficiency. There is also the possibility of error under such conditions.

Water tanks should be preferably of the automatic volumetric type or of the automatic weighing type. The automatic weighing type with electric controls is a practicable unit. There are a number of devices on the market that have been developed by various manufacturers for this operation.

The concrete mixers should be equipped with power discharge mechanism, within easy reach of the operator. They should also be equipped with a batchmeter and locking device such as is now in general use on paving mixers, so that the mixing time can be controlled properly.

The water pipe lines should be of sufficiently large size to permit rapid discharge of water into the mixer at the proper time. It is important that sufficient water enter the mixer to increase the mixing action and to prevent the building up of materials on the inside of the drum.

CENTRAL PROPORTIONING PLANTS—TRUCK MIXER DELIVERY

As previously stated, the operation of this plant is generally the same as that of the wet mix plant, with the exception that stationary mixers are not used, the material being charged into a truck body which mixes enroute.

THE COMBINATION PLANT

In this type of plant provision has been made for mixing at the plant and discharging into agitator type trucks, or proportioning for mixer trucks, or for dry batching materials for use in road pavers.

The general design of this type of plant is essentially as described above under "Wet Plants" but with by-pass arrangement controlled by the operator to divert the flow of materials into the central plant mixer, or directly into the truck mixer, or for loading batch trucks. The cost of the combination plant is very slightly more than that of the wet plant.

It is necessary to provide for heating the materials in the aggregate storage bin for cold weather operation. This is best done by live steam from a high pressure boiler with jets introduced into the lower section of the bin or with coils arranged inside the bin itself. The water used in mixing, must also be heated before passing through the water measuring tanks. The use of an auxiliary tank for heating and storing water is recommended to meet this requirement.

It is essential that devices be used in central proportioning plants to control the flow of materials so that the cement and various aggregates are fed uniformly into the opening of the truck mixer. This arrangement affords the advantage of a partial premix of the concrete materials and is a distinct advantage in the production of better concrete.

DELIVERY OF CONCRETE

Pre-Mixed Concrete, the Agitator, Semi-Agitator and Plain Trucks

After the sand, stone, cement, etc., have been deposited in the

mixer and been properly mixed, it is discharged into various types of agitating bodies.

1. The cylindrical drum type of agitator, is charged through a hatch at the top and the cylinder is revolved enroute to the job. This body is elevated or tilted by a standard truck hoist for the discharge of materials, through a rack and pinion tail gate. This unit is provided with an independent gasoline power plant for revolving the cylinder.

2. An open top agitating body has a central shaft with paddles attached. This type of truck is elevated by standard truck hoist to an angle of approximately 50 deg. and the discharge of concrete is by gravity and controlled by a rack and pinion tail gate. The agitation is controlled by power take-off gear arrangement from the truck engine.

3. A cylindrical type of body has blades on the inside either of the bucket type or of screw conveyor type, by which the material is propelled to the rear of the truck and discharged through rear control door. These units have an independent power plant for revolving the body. They require no hoist.

4. From another type the material is discharged from the bottom of a cylindrical body onto a pan where it flows by gravity to the rear of the truck.

5. A bathtub type of open top body has no agitator equipment of any kind, and the concrete is discharged in much the same manner as from the ordinary material dump truck.

6. The remixing type of open top body is intended to give a partial remixing action of the concrete when the body is tilted and discharged. This unit also has rear end control gate with rack and pinion arrangement.

7. Finally, there is the plain open-top, dump-truck body with truck-hoist and rear control, tail gates.

It is highly important that the concrete mixer be of the same capacity as the capacity of the delivery unit. This eliminates lost time and makes for general efficiency in the operation of the plant.

Truck Mixers

The equipment for this type of plant is practically the same as for wet mix plants, but with the addition of special loading devices designed to meet the requirements of the type of mixer truck

that is being used. Three types of truck mixers now in general use are as follows:

1. One type is a cylindrical body operated through a power take-off and gear drive from the engine of the truck. In this unit the material is charged into the cone-shaped body through an opening at the rear, and the proper amount of water added while mixing, enroute to the point of final deposit or after the truck arrives at the job. The water is measured at the plant and deposited into a tank having two compartments, one which contains the measured amount of water for the mixing of the concrete proper and the second smaller compartment which contains a small amount of water used in washing the truck out on its return trip.

The discharge of the concrete from this type of body is effected by the tilting of the body through a mechanical type truck hoist, the body revolving while discharging. While the truck is enroute to the job, the opening at the end of the drum is covered with a removable door to prevent leakage or loss of material.

2. Another type of truck mixer has a drum with a series of buckets and blades, and a discharge chute. This truck is charged at the plant through an opening in the top of the drum and the cover is then closed. Operation of the drum is by an auxiliary gasoline engine operating through a series of clutches to reverse the action of the mixing drum, for the purpose of mixing or discharging. This unit is equipped with a water tank properly calibrated, and the amount of water is added by the operator of the truck, in accordance with the required specifications. This tank has two compartments, one for mixing water and one for flushing or cleaning water.

The discharge of the concrete in this unit is effected by the removal of the cylindrical cover at the rear of the drum, and the introduction of a discharge chute similar to that used in the regular drum type concrete mixer.

3. Another truck mixer unit is also designed to be loaded through a removable cover at the top of the drum. This cover is closed when the unit has been loaded and the material is then mixed by rotating the drum. When the unit arrives at its destination, it is revolved in the opposite direction, thereby bringing the

material to the rear and discharging it through a cylindrical gate with wheel control.

This unit is also equipped with a calibrated water tank, the water being added by the operator of the truck to meet the required specifications. This design of truck unit with the interior blading arrangement similar to the screw conveyor, is equipped with an independent power plant and also with a power takeoff from the truck motor itself, with a series of properly designed reduction gears.

All of the above type agitator bodies and mixing truck bodies are built in sizes of one-to six-cu. yd. capacity and are generally mounted on trucks designed to carry the necessary loads. They are equipped with pneumatic tires and solid rubber tires, to meet local conditions.

There are various types of trucks now in use for this purpose, ranging from the regular four wheel trucks to the six wheel trucks, and the four wheel trailers.

The number of truck units necessary for the proper operation of the plant is determined entirely by the amount of concrete to be delivered and the length of haul. In some cases, a number of producers of ready mixed concrete have combined their operations in a manner that permits the transfer of a number of trucks from one plant to another, so as to take advantage of peak load requirements. This has worked economically in a number of instances.

MISCELLANEOUS PROBLEMS

In some plants individual problems have been solved by the ingenuity of the operators in devising special equipment and methods to meet the particular requirements of their local conditions, and it is safe to state that a majority of present plants are operating successfully but probably do not represent the ultimate development in plant equipment, plant control, or hauling and placing methods.

It is strongly advised that proper engineering control be set up covering all of the factors entering into the manufacture and delivery of central plant concrete. This should include proper inspection and testing service. Facilities should include the determination of proper concrete aggregates and their inspection. Suitable equipment should be provided at the plant for determining the moisture content of the fine and coarse aggregates,

and tests of this condition shall be made a number of times each day. Suitable engineering service should be available for the proper design of the various concrete mixtures required. Provision should be made for frequent inspection of all the measuring equipment used in the plant to insure constant accuracy.

There are a number of methods in use at the various plants throughout the country for records of maintenance costs, methods of keeping material receipts, delivery receipts, etc., all of which this Committee is assembling for a future report.

It is also desirable in all of these plants that contact be maintained between the office and the operator. This is best done through the use of a pneumatic system or some form of electric signal for delivering messages. The use of a telephone on the platform particularly in a wet mixed plant is not generally efficient, because of the noises from the mixer, motor, etc., which interfere with the clear hearing of the operator and result in misunderstandings. There are electric light systems that are used in some localities which have been effective in transmitting information from the office to the operator.

It is suggested that a machine shop be maintained at every plant where repair parts and replacements are carried for the truck equipment, and where necessary replacement items shall be kept on hand in case of wear or breakdown of any part of the plant itself. A few extra parts such as gears, chain, fuses, etc., will save their cost many times in the course of a year, by preventing shutdowns in the plant operation.

This report obviously is not complete but is intended to stimulate interest and discussion. There is in the hands of the Committee a considerable amount of detailed information in an incomplete state, which we will endeavor to compile and report on as rapidly as possible. Much research work is now being done along constructive lines and additional data on the subject will be submitted to the Institute in a later report.

Readers are referred to the JOURNAL *for December 1931 (Vol. 28), for discussion which may develop. Such discussion should reach the Secretary by Nov. 1, 1931.*

Specification for Concrete Burial Vaults

*Progress Report of Committee 709**

W. D. M. ALLAN, CHAIRMAN

INTRODUCTION

November 12, 1930, Wilber M. Krieger, executive secretary of the National Selected Morticians, wrote the secretary requesting that the American Concrete Institute prepare a specification which could be used to measure the quality of concrete burial vaults. At the same time Mr. Krieger wrote H. A. Ledyard, president of the National Concrete Burial Vault Association, outlining the value of a specification prepared by a national technical organization in extending the use of this product.

Department of Commerce reports show that the output of concrete vaults for 1929 was approximately 80,000 but members of the committee believe that actual production is nearer 125,000. The total vault business available for all types and materials is about one million per year. Even though the vault field is far removed from ordinary concrete work the satisfactory performance of the finished product depends upon the quality of concrete used. It is therefore proper that the Institute recognize the problem and comply with the request to develop, if possible, a satisfactory specification.

Attention has been directed by many vaults of poor quality and by the advertising of competitive types of vaults to the necessity for a standard specification. This led to the formation of Committee 709.

For nearly two years the National Concrete Burial Vault Association has had under consideration a specification for con-

*Presented at the 27th Annual Convention, A. C. I., Milwaukee, Feb. 24-26, 1931 by C. A. Wiepking, Testing Engineer, Milwaukee, Wis., member of the Committee. Other members of the Committee are: C. L. Berg, R. F. Hanneman, Prof. Carl Head, H. A. Ledyard, F. J. Mead, George M. Painter, H. G. Samson, J. H. Stuart and F. H. Van Ness.

crete vaults. Data collected by their research committee has been turned over to this committee.

In order to understand the quality of concrete required to make a satisfactory vault, it was necessary to study the problem from the standpoint of the funeral director, the cemetery superintendent and the manufacturer. The funeral director requires that the concrete be watertight, durable, strong, and dense enough to resist attacks of chemicals in ground waters. The cemetery superintendent demands the same factors but is especially interested in strength requirements which will eliminate sunken graves. The manufacturer requires a container of minimum weight that will meet the purchaser's requirements and which can be manufactured at a cost to compete with other types of vaults. The specification must cover all of these points and still be simple and easy to enforce.

Time has not been available since the formation of the Committee Jan. 20, 1931, to present a Tentative Specification. The Committee believes that this report should be on record so that discussion can be drawn from morticians and vault manufacturers on the provisions of the specification and the possibility of enforcement.

SUGGESTED SPECIFICATION FOR CONCRETE BURIAL VAULTS

1. GENERAL

Scope

1. This specification applies to reinforced concrete vaults, intended for use as burial receptacles, made in two pieces; a base and cover.

Basis of Acceptance

2. The acceptability of vaults shall be determined by the results of the strength, absorption and hydrostatic tests herein after specified, and by visual inspection.

Discussion—For well established plants with rigid methods of control, it is suggested that visual inspection and strength and absorption tests on cylinders should be sufficient to insure the quality of the vault, providing the purchaser is satisfied with methods for sealing at the grave.

2. MATERIALS

Reinforced Concrete

3. The reinforced concrete shall consist of portland cement, mineral aggregate and water in which steel reinforcement has been embedded.

Cement

4. Portland cement shall meet the requirements of the current Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.

Steel

5. Reinforcement shall consist of wire which meets the requirements of the current Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement of the American Society for Testing Materials, or of bars which meet the requirements of the current Standard Specifications for Billet-Steel Concrete Reinforcement Bars of the American Society for Testing Materials.

Aggregates

(a) Fine aggregate shall consist of sand, stone or slag screenings, or other inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam or other deleterious substances. Fine aggregate shall be well graded and shall pass a ¼-in. screen. At least 5 and not more than 20 per cent shall pass a 50 mesh screen and at least 10 and not more than 20 per cent shall be retained on a No. 8 screen.

(b) Coarse aggregate shall consist of crushed stone, gravel, slag, or other approved inert materials with similar characteristics, or combinations thereof, having clean, hard, strong, durable, uncoated particles, free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter. All of the coarse aggregate shall pass a ⅜-in. screen and be retained on a ⅛-in. screen.

3. DESIGN

Minimum Design

9. The vaults shall be designed in accordance with the following assumptions:

(a) That the design load is equivalent to the vertical load uniformly distributed over the internal horizontal cross-sectional area of the vault and that the vault is likewise uniformly supported.

(b) The distance from the center of the reinforcement to the nearest surface of the concrete shall not be less than (. . . .) in.

Discussion—This dimension has been left blank because it depends upon the minimum thickness of vaults permitted. In general only one line of reinforcement is used so that this distance should be one-half the minimum thickness of vault decided upon.

(c) The shell thickness of the vault shall not be less than (. . . .) in.

Discussion—The shell thickness finally decided upon must consider the manufacturer's demand for minimum weight, yet provide for adequate protection for reinforcement and resistance to normal earth and water pressures. The Committee has not determined as yet whether this should be one inch, one and one-half inches or two inches.

(d) The minimum weight of reinforcement shall be equal to (. . . .) lb. per 100 sq. ft. of vault wall area and if of mesh shall have openings at least (. . . .) in. square.

Discussion—The average weight of reinforcement now used is approximately 50 lb. per 100 sq. ft. but individual manufacturers vary widely from this average. The opening in mesh must be large enough to permit the concrete to pass freely through and embed the individual members. The larger mesh also permits the use of heavier wires, is easier to handle and is frequently more economical. A minimum size opening 2 in. square and a maximum of 6 in. square are being considered.

(e) The net quantity of mixing water in the concrete mixture, including the surface water carried by the aggregate shall not exceed (. . . .) gal. per sack (94 lb. net) of cement. The proportions of aggregates to cement shall be such as produce concrete that will work readily in the form and around reinforcement without excessive puddling or spading and without permitting the materials to segregate or free water to collect on the surface. The slump of this concrete when measured according to the current Method of Test for Consistency of Portland Cement Concrete of the American Society for Testing Materials shall be not more than (. . . .) in.

Discussion—Experiments conducted over a period of two years at the Portland Cement Association Laboratory show that in thin sections (1 to 2 in.) with 7 days moist curing not more than 5 gallons of mixing water net should be used per sack of cement to produce concrete of unquestioned watertightness. The suggested maximum slump 4 to 5 in. is being considered to prevent segregation of aggregates and the tendency to form water pockets.

Placing Reinforcement

12. When a single line of reinforcement is used, it shall be placed in the center of the wall. The reinforcement shall be lapped not less than 30 diameters if of bars or 2 openings if of

mesh and securely tied with No. 18 annealed wire, or if welded the joints shall develop the full strength of the reinforcement.

4. WORKMANSHIP AND FINISH

Finish of Concrete

13. Vaults shall be smooth, free from stone, air or water pockets and structural cracks.

5. MARKINGS

Markings

14. All vaults made in accordance with this specification shall be marked as follows:

a. Meets current A. C. I. Specification

b. The name or identifying mark of manufacturer

6. PHYSICAL TESTS

Concrete Tests

15. The quality of concrete shall be determined by compression and absorption tests on 3 by 6-in. cylinders taken from the concrete used in making the vaults and manufactured and cured under identical conditions with the vaults.

Compression Tests

(a) When tested in accordance with the current standard methods prescribed by the American Society for Testing Materials, these cylinders shall have a strength of not less than (. . . .) lb. per sq. in. when 28 days old or when the vaults are delivered, if delivered in less than 28 days.

Discussion—The strength requirement has been usually considered as 4,000 lb. per sq. in., but since watertightness is a major factor in producing satisfactory vaults, it may be that in order to get the degree of watertightness necessary, the strength will be automatically increased to 5,000 or 6,000 lb. per sq. in. A higher compressive strength is also necessary if the thinner walls suggested in this report are adopted.

Absorption Tests

(a) At least three 3 by 6-in. cylinders in addition to those submitted for the compression test shall be furnished for the absorption test. The specimens shall be indelibly marked, to identify them with the vaults made with the same batches of concrete and cured in the same manner as the vaults.

(b) Specimens shall be dried at a temperature of from 100° to 110° C. (212° to 230° F.) for 24 hours.

Discussion—Laboratory tests show that 3 by 6-in. cylinders made of 5 gallon concrete and saturated at the beginning of the test will lose within 1.25 per cent of its total loss in weight with 24 hours drying at the temperatures specified. Since it will be only in prolonged rainy seasons in the case of vaults stored in the open that the specimen will be saturated, the 24-hour drying is considered sufficient. This is especially true, since the loss on drying at these temperatures for 24 hours will cause the concrete to lose more water than it would under any usual atmospheric condition. By decreasing the absorption requirement one per cent, the test for 24 hours can be corrected to equal complete drying.

(c) The dried specimens shall be placed in a suitable receptacle, covered with distilled water or rain water at room temperature (70 deg. F.) for 24 hours. After this immersion period, the specimens shall be removed from the water, allowed to drain for not more than one minute, the superficial water removed by a towel or blotting paper, and the specimen immediately weighed.

(d) The balance used shall be sensitive to 0.5 gr. when loaded with 1 kg. and weighings shall be read at least to the nearest gram. Where other than metric weights are used, the same degree of accuracy must be obtained.

(e) The increase in weight of the immersed specimen over its dry weight shall be considered the absorption of the specimen and shall be calculated as a percentage of the dry weight by the following formula:

$$\text{Percentage absorption} = 100\,\frac{\text{weight wet} - \text{weight dried}}{\text{weight dried}}.$$

The results shall be reported separately for each specimen.

(f) The average absorption shall not exceed (....) per cent for three (3) test specimens with no one specimen testing more than (....) per cent.

Discussion—It has been suggested that the average absorption limit should be placed at 3 or 5 per cent with no one specimen testing more than one per cent above the average agreed on.

Strength Test on Vaults

16. Vaults shall be tested with a load applied on top equal to (....) lb. per sq. ft. of inside horizontal cross-sectional area.

(a) The vault shall be set on a uniform bed of moist sand of such size that all of the particles will pass a ¼-in. screen.

(b) The top bearing frame shall not be allowed to come in contact with the top bearing plate. The upper surface of the sand

in the top bearing frame shall be struck level with a straight edge, and shall be covered with a rigid top bearing plate, with lower surface a true plane, made of heavy timbers or other rigid material, capable of distributing the test load uniformly without appreciable bending. The test load shall be applied uniformly over the top of the vault. (See Fig. 1.)

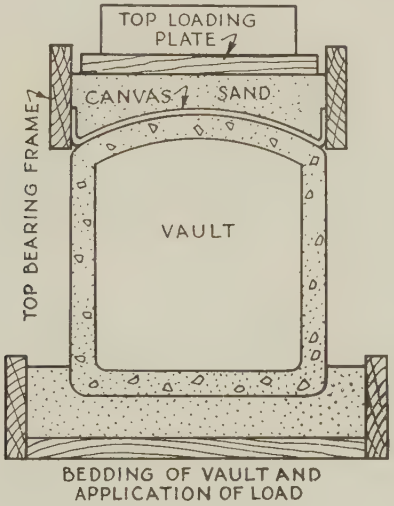


FIG. 1—SAND BEARING FOR COMPRESSION TEST ON CONCRETE BURIAL VAULTS

(c) The top and bottom bearing frames shall be made of timbers heavy enough to avoid appreciable bending by the side pressure of sand. The interior surfaces of the frames shall be dressed.

Discussion—The maximum puddled earth load on the vault in service is approximately 390 lbs. per sq. ft. of inside horizontal cross-sectional area but the committee believes that the test load should be increased to provide a factor of safety. Five hundred to six hundred lbs. per sq. ft. have been suggested. Field tests of vaults made under this specification will be made to determine what this load should be.

By using slabs of concrete having a combined weight equal to the required load minus the weight of the upper frame, sand and bearing plate, the test may be made very quickly on each vault sold or as required. Hydraulic jacks set up in rigid structural steel frames may also be used. (See Fig. 2.)

(d) After the load has remained on the vault one hour it shall be removed and the specimen inspected. The vault shall have failed if any visible structural cracks have developed during the test.

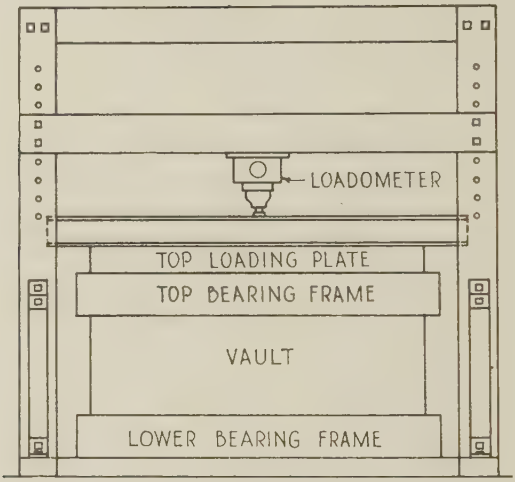


FIG. 2—SAND BEARING WITH LOAD APPLIED BY HYDRAULIC JACK

(e) At least one loading test must be made by the manufacturer each month but the purchaser may require one test be made for each (. . . .) vaults purchased.

Discussion—The strength of the test cylinders are a close indication of the strength of the vault manufactured according to this specification, and it is recommended that the purchaser accept the results of the single vault test made each month by the manufacturer together with the compression and absorption tests on cylinders as a measure of the vault strength. The suggestion has been made by some, that the purchaser be permitted to require one loading test for each ten vaults purchased.

(f) All vaults for testing purposes shall be selected at random by the purchaser or his representative from the stock of the manufacturer and shall be vaults which would not otherwise be rejected under this specification.

Hydrostatic Test

17.

Discussion—The method of making the hydrostatic test has not been developed to a point where agreement is possible. A concrete pit six feet deep in which the vault may be submerged and held down by a suitable weight or

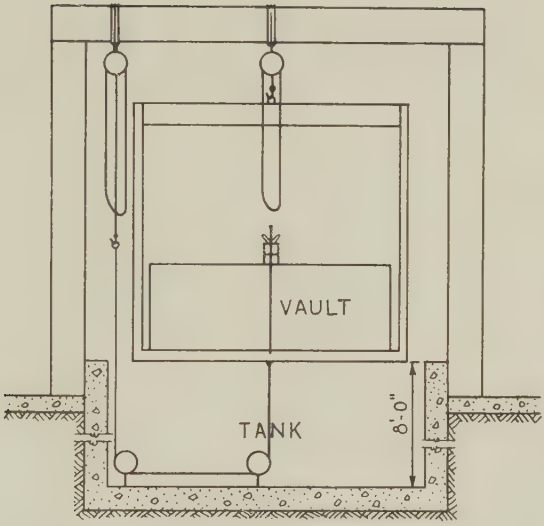


FIG. 3—HYDROSTATIC TEST BY SUBMERSION

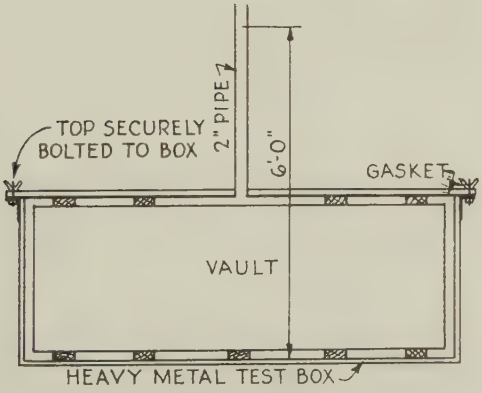


FIG. 4—HYDROSTATIC TEST BY PRESSURE BOX

system of winches has been used by a number of manufacturers. A rigid container equipped with a standpipe of suitable height in which the vault could be placed has the advantage of simplicity. A disc of suitable area, equipped with a two-inch standpipe, which could be fastened to the side of the vault is relatively cheap and easy to apply but has the disadvantage of not testing the entire vault including the seal. Discussion on this test will be particularly welcomed. (See Figs. 3, 4, 5.)

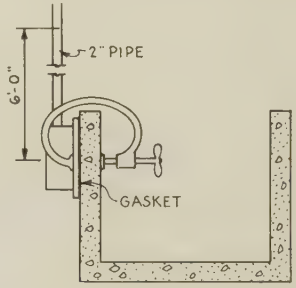


FIG. 5—HYDROSTATIC TEST ON SMALL AREA

Minimum Age for Shipment

18. Vaults will be considered ready for shipment when 28 days old, or when cylinder tests indicate that the concrete has attained the strength required.

7. SEALING

Materials and Methods

19.

Discussion—The Committee has not had sufficient time to investigate this subject and welcomes discussion by all interested parties. It has not, however, been definitely decided that this subject should be included directly in the specification. It will, of course, be included indirectly by the hydrostatic test.

8. INSPECTION

20. All materials, processes of manufacture and finished vaults shall be subject to inspection and approval by the purchaser or his representative.

21. Vaults shall be subject to rejection on account of the failure to meet any of the specification requirements.

END OF SPECIFICATION

The Committee presents its report in a form intended to create discussion and invites manufacturers, purchasers and interested engineers to review these requirements and submit their discussions.

Readers are referred to the JOURNAL *for Dec. 1931 (Vol. 28), for discussion which may develop. Such discussion should reach the Secretary by Nov. 1.*

Discussion of Report of Committee 203

"Cinders as Concrete Aggregate"*

CONVENTION PRESENTATION BY EINAR CHRISTENSEN, AUTHOR-CHAIRMAN

It makes me very happy that the American Concrete Institute has set aside this session for some of the problems of light weight concrete. It indicates that we are now in a position to spend some time on matters not pertaining to the dams and the beams and the arches and the roads that consume nine-tenths or more of our good cement—and can begin to consider the field beyond, which at present is small but which I believe will be of increasingly great importance.

Light weight concrete has been with us for some time—and I do not think anyone will question that it is here to stay. Whether the ultimate light weight aggregate is cinders (as they can be, and as I expect to see them prepared), or it is Haydite or some other natural or artificial cellular material, is something I do not know. Purpose, quality and economy must be considered. But I think that the lines of development have been laid out.

It should not be presumptuous to say that the cinder concrete industry took the early lead. It was through study of cinder concrete that we discovered that concrete might have other attributes besides those of strength and density. We began to hear about lightness, insulation, nailability, etc. I am not much wrong when I say that cinder concrete has been the step-child of the concrete industry. In saying that, I am not complaining. It is only during the last ten or twenty years that we have gotten down to fundamentals of rational design of stone concrete—and that has naturally and rightly kept the Institute busy.

*A. C. I. Journal, Feb. 1931; *Proceedings*, Vol. 27, p. 583. Mr. Christensen as Author-Chairman of the Committee made a presentation of the report at the 27th Annual Convention, Milwaukee, Feb. 24-26, 1931. Critic members of the Committee are: W. D. M. Allan, R. D. Bradbury, Miles N. Clair, Emil Praeger, M. J. Roach. The discussion here is incomplete and will be concluded in the December, 1931, Journal, Vol. 28.

It is my conviction that time is now ripe for a survey of our knowledge of cinders as an aggregate for concrete. The grand objection to cinder concrete has been that the material could not be put on a basis of rational design—but this is no longer true of factory-made cinder concrete, and it need not be true of job-made concrete. Perhaps we know a little more about the subject than some members think. It was to find out how much we know that Committee 203 was appointed.

It was really a big assignment—to cover the whole field from red hot cinders on the grate to the calm and presumably contented concrete in the structure. The result is in the February JOURNAL. As an easily read paper the report suffers from being much too long—as a comprehensive report it suffers by being much too short. Brevity is likely to be attained at the expense of some accuracy, which is most unfortunate, and it frequently leads to a confusion of opinion and fully established fact. How far we have avoided that in the report is something I cannot fully judge.

In reading the report, you will probably miss that favorite statement of many authors, that "more research is needed here, there and everywhere." I did not find it necessary to say that over and over again—the need for more research should be apparent to every reader. I also hope that the report shows that such research is not only needed but appears justified and deserved by what may be accomplished. I shall feel very much disappointed indeed if some of the opinions advanced in the report are not disproven or altered within the next ten years. But if the report can bring about such learning of new facts, then it has accomplished its main purpose.

In giving a brief synopsis of the paper I realize the difficulty of condensing these sixty-odd pages into ten minutes—particularly when considering the fact that the whole report as it now appears was a matter of considerable condensing. But I shall try.

Synopsis

Certain definitions are used throughout the report. For example, the term "Cinder Concrete" is applied only to a concrete made with an aggregate of nothing but cinders, while the term "Sand-Cinder Concrete" is applied to the type of concrete in which the aggregate is composed of both sand and cinders. "Cinders" is differentiated from "Ashes"—the former term applying to the residue from high temperature combustion of coal or coke, while the latter

applies as a more inclusive term, to all residues from the incomplete combustion of carbonaceous matter.

No differentiation is made between anthracite and bituminous cinders. While they differ in their physical character, they are both suitable for concrete when meeting the requirements here proposed for the raw material.

Cinders—The constituents of cinders affecting the value of the material as an aggregate for concrete are coal, coke, sulphates, sulphides, and (perhaps) lime.

The effect of combustible matter on fire resistance is surprisingly small. The Underwriters' Laboratories, in their present specifications for hollow concrete building units, state: "When cinders are used the average combustible content of the mixed fine and coarse aggregate shall not exceed 35 per cent by weight of the dry mixed aggregates." Bituminous coal of a certain character (having high absorptive power) may cause expansional movement during the early life of the concrete. A soundness test is described.

Cinders usually contain a small amount of sulphur compounds (sulphates, sulphides, etc.). Corrosion of steel in cinder concrete occurs only when the concrete is of such poor quality that air and moisture readily get into contact with the steel. Under the same conditions, corrosion will occur in concretes made with any type of aggregate. It is possible that in such concrete the corrosion of the unprotected steel will be accelerated by the presence of sulphates and the like in solution, but it is absurd to ascribe any effect as due to "oxidation of sulphides to sulphuric acid." The uniformly successful record of sand-cinder concrete in New York, as far as protection of steel is concerned, cannot be ignored.

Specification for Cinders—A specification for cinders should be liberal enough to be practical; otherwise it is not enforced. Based on this idea the report proposes a specification simply calling for cinders (not ashes) from high temperature combustion, well burned, free from foreign matter and so graded as to produce a cinder concrete meeting the strength requirements of the building code. Based on certain tests, limits for combustible matter and sulphur compounds are proposed.

Judging Cinders for Concrete—Judgment is still an important factor in making concrete, and when cinders are used as an aggregate it is well to have some simple rules for judging the material without having to resort to elaborate physical and chemical tests.

The report proposes certain rules. First, something should be found out about the source of supply; frequently records are kept there of analysis of cinders. Second, if the cinders contain rubbish they should not be used because the pile may have been a general dump and the cinders may contain dangerous impurities. Cinders from a chemical plant, it may be well to test with a water extract (boiled with distilled water) with litmus paper; the paper should turn slightly blue. Third, representative sample of the cinders should be dumped into a bucket filled with water; the fine ash and the lighter coke will float and the amount and nature of these may be judged. After draining the water from the bucket the appearance of the washed cinders should be compared with

that of the unwashed; the knowledge is gained of the relative amount of coke, coal, fines, porous, flaky, or fully vitrified clinker; the grading may be observed. Fourth, the strength of the larger particles may be judged by breaking them down. The weight per cubic foot should be determined.

The use of the bucket and the other simple methods proposed may appear very crude to the engineer—nevertheless, such simple methods have been found very useful, when used with some experience and common sense.

Testing Cinders for Concrete—When it is a matter of selecting a large and steady supply of cinders, a definite effort should be made to obtain the raw material from one known and reliable source. The cinders should then be subjected to more exhaustive tests, over enough time to determine average quality.

The tests should include determination of combustible content, nature of coal, soundness test of coal if bituminous, examination for impurities, sieve analysis, etc.

Making Cinder Concrete Building Units—The typical unit of the cinder unit industry is the 8 x 8 x 16 in. hollow block, containing two or three hollow cores, or from 30 to 40 per cent hollow space. Most specifications require this unit to have a compressive strength, at 28 days, of 700 lbs. per square inch of gross area.

Strength is not the sole criterion for quality of cinder concrete; the cellular structure should be developed in order to obtain such characteristics as lightness, insulation, nailability, etc. The resulting concrete should be considered within its own field, not as a general substitute for stone concrete.

Maximum size of aggregate is generally limited to ½ to ¾ in. Most desirable grading appears to be represented by a fineness modulus of 4.0 to 4.5 (maximum size ¾ in.). A simple rule suggests that the aggregate "split" 50-50 or 40-60 on ¼ in. sieve, assuming usual range from coarse to fine as found in crushed but unscreened cinders.

The voids of loosely packed aggregate appear to average about 55 per cent, with from 8 to 16 per cent inaccessible, sealed cells. A large amount of moisture is generally present in raw cinders when received. Damp volume is greater than dry volume, a bulking of about 25 per cent having been recorded, fairly constant within wide range.

Cinder concrete masonry units are made from a relatively dry mix, under pressure. It is well to bear this in mind when comparing the material with job-poured concrete; while machine made cinder concrete lacks plasticity, it is placed in such a way that homogeneity and the necessary density may well be obtained. Total mixing water necessary depends upon material and conditions. Tests are recorded representing best results at 1.5 to 1.8 gallons of water per cubic foot of aggregate (for 1:7 to 1:10 mix).

Open stationary-drum mixers are used. Conveyers for cinders are designed to minimize abrasive effect of material. Crushers of the jaw-type, slow-hammer type, or rolls are used in primary reduction; rolls are best for secondary reduction. Grading of cinders, while highly desirable, is at best an incomplete operation due to dampness of cinders and shape of particles. Only fast vibrat-

ing screens are suitable. It is important to prevent segregation of aggregate; this is attained by having storage bins of small diameter, keeping them full, or by dumping cinders in bulk.

Making Sand-Cinder Concrete—Sand-cinder concrete should be designed for the purpose for which it is used, and not according to arbitrary specifications like the "1:2:5" so frequently encountered. The main reason for the use of cinders in this type of concrete (structural concrete) is the lightness of the aggregate. The main reason for the addition of sand is to produce workability.

Tests indicate that blindly adding sand does not of necessity increase the strength or improve the quality of sand-cinder concrete; the grading of the combined aggregates must be considered. High uniform strength tests are reported on 1:2:5 and 1:2:4 sand-cinder concretes where the cinders had a fineness modulus of about 5.7, the sand about 2.4, and the mixed aggregates about 4.65.

In the same series of tests the total volume of mixing water (including that present in the damp cinders when received) measured about 13 to 14 gallons per sack of cement for the 1:2:5 mix and about 11 to 12 gallons for the 1:2:4 mix. The concrete was used in casting reinforced beams.

Recent tests indicate that cinder concrete without sand may give excellent results for structural purposes.

It is believed that there should be a considerable market for a prepared cinder, crushed and graded, and properly mixed with sand if necessary, to meet definite requirements. This may be the only way in which rationally designed mixtures may be obtained for structural purposes.

Characteristics of Resulting Concrete—The usual strength requirement for cinder concrete building units (hollow) is 700 lbs. per square inch of gross area. The compressive strength of standard 8 x 16 in. cylinders of good New York sand-cinder concrete is reported to be 800 to 1,000 lbs. per square inch at 28 days (1:2:5 mix), the modulus of elasticity is about 1,200,000 lbs. per square inch. Much higher results are obtained under conditions of better control than that usually found on jobs at present.

The average weight of cinder concrete, as used in the cinder unit industry, appears to be about 85 to 90 lbs. per cubic foot. 1:2:5 sand-cinder concrete (structural concrete) averages about 110 to 115 lbs. per cubic foot.

When comparing absorption percentages with those of stone concrete and other heavier building materials, consideration should be given to the lightness of cinder and sand-cinder concrete. The significance of the usual absorption test is questioned. The test does not seem to give information regarding resistance to weather and freezing, capillary behavior, etc. The shape, size and distribution of pores should be studied.

The fire resistance of well made cinder and sand-cinder concrete appears well established. The behavior under exposure to fire and water is remarkable; the heat transfer is low when the cellular structure is properly controlled.

Data on cinder concrete (as used in making cinder units) indicate the Coefficient of Conductivity "K" to be 3.33 (per inch thickness). For usual

conditions, an 8 in. hollow cinder block wall (exposed) will have a Conductance (C) of 0.45 or less, and a Transmittance (U) of about 0.34.

Exposed surfaces of cinder concrete appear to have a very high sound absorption. The sound transmission through properly constructed walls of cinder concrete appears low. As with heat transmission, the cellular structure is the determining factor.

Properly made cinder concrete will receive and hold nails, and woodwork is nearly always nailed direct to the masonry wall. Machine made (relatively dry mix) cinder concrete building units have an interesting surface texture offering remarkable adhesion for plaster, mortar and the like.

Such is our report on an aggregate that some able engineers use to the extent of millions of cubic yards, while other able engineers flatly condemn it. They did not wait for this review to build the Empire State Building, the Chrysler Building or all the other buildings that have given New York its vertical growth, and in which so many thousand yards of cinders were used. I don't think the report will make them regret it. Rather, I hope the report will lead to an increasing and an increasingly intelligent use of cinders as an aggregate for concrete.

In getting up the report I have received great help from the members appointed on Committee 203. In this respect I am probably much more fortunate than some other author chairmen—and I am very grateful.

WRITTEN DISCUSSION

BY GEORGE E. STREHAN*

The author's proposed definitions of "cinder concrete" and "sand-cinder concrete" are not in line with engineering practice and in the writer's opinion are misleading. Portland cement concretes are generally designated as "stone concrete," "slag concrete," "gravel concrete," etc., as determined by the coarse aggregate. The discussion advanced in the report does not warrant a change from this practice. In the writer's opinion, what Mr. Christensen refers to as cinder concrete in the "factory-made cinder unit" of today is not well-made cinder concrete at all. Plasticity, workability and durability are generally required essentials of good concrete. Plasticity depends upon the presence of a sufficient quantity of plastic paste to form a homogeneous

*Consulting Engineer, New York City.

mixture with the coarse aggregate; workability determines the ease of combining and placing the mixture without excessive voids, honeycombing or segregation; and durability in concrete construction exposed directly or indirectly to the elements is in large measure dependent upon the absence of porous or cellular structure.

The two controversial features connected with the use of cinder concrete, whether in the form of precast shop-made units or monolithic, poured-in-place concrete are: (1) corrosion of reinforcement; and (2) durability. The writer would dismiss all discussion of the fire resistance of cinder concrete as having been demonstrated without a margin of doubt both in practice and laboratory experience. The controversies as regards corrosion of embedded steel and the permanence of cinder concrete are however, exceedingly live subjects; and are due to the *misuse* of cinder concrete rather than to any inherent defect of the material itself. The author refers to the excellent behavior of the cinder concrete construction in Professor Woolson's test house erected in 1905 at Greenpoint, Brooklyn, which is still doing duty as a test furnace after some 300 to 400 hours of 1700 deg. F. fire. The walls of this structure were made of a 1:2:5 plastic mix of sand, cement and steam boiler plant cinders quite different from the concrete generally employed for cinder block units.

Mr. Christensen speaks with authority on the subject of the shop-made cinder building unit. He is concise and thorough on the subject of selection of proper cinders for pre-cast units. His seven rules of procedure to determine the quality of cinders for use as an aggregate should provide a good measure of the quality of the material for aggregate purposes in present day practice. The requirements of practical value are maximum permissible combustible content in the cinders (now fixed by the Underwriters at 35 per cent), and minimum concrete strength requirements for use of the unit for either load bearing or non-load bearing purposes. The discussion of the manufacture of cinder units is thorough in detail and presents an ideal for the industry. The fact remains however that to the average block-plant, cinders that can be bought for 50 cents a yard are vastly more attractive than cinders costing $1.40 a yard for fixed grading and correct chemical composition. One very important considera-

tion is not stressed sufficiently by the author. The machine-tamped cinder concrete unit is made of a relatively dry, non-plastic mixture. Sufficient water cannot be added to the mixture for complete hydration without encountering the possibility of slumping of the block. The curing conditions prevailing in most cinder concrete products plants are not conducive to complete hydration. Accelerated or steam curing is seldom resorted to. A liberal use of calcium chloride is a frequent substitute used to accelerate setting. The author states that "Because of the porosity of the cinder aggregates, a large water reserve is available for hydration of the cement, and because of the nature of the concrete, the water is slow in reaching the surface where it evaporates." This is contrary to the writer's experience. Because of the physical characteristics of the aggregate, the excess water in the block has a tendency to rapid drainage.

It is known that well-made plastic concrete deposited on the interior of a building exposed to extreme dry heat will lose strength in time, through the arrested process of hydration caused by the absence of the necessary moisture content. It is the writer's belief that very little study is given to the control of water content in the average cinder unit plant; and frequently the only water used in manufacture is the water contained in the cinder aggregate as stored amounting perhaps to 4 or 5 lbs. of water per cubic foot. The uncured cinder unit placed in the wall continues to hydrate for a considerable period beyond the conventional 28 days, accompanied by volumetric changes that are always incident to this continued "working." This process plus the cellular character of the block unit, in the writer's opinion, accounts for much of the lack of durability or permanence in the structure as well as the corrosion of embedded steel occasionally demonstrated by the rusting of nails driven into the masonry. Moisture absorption is particularly large in cinder mortars and in masonry walls, the cinder concrete building unit is not fully protected against the moisture forced in from the outside. The ultimate curing and hydration of the cement content in the block is dependent on the acquisition of this moisture. The high expansional movement of cinder concrete normally associated with its early life is very likely to be prolonged to the

masonry in the wall because of the slow hydration of an all-cinder concrete mixture.

The second sub-division of the paper dealing with the practices and uses of "sand-cinder concrete," has failed to emphasize some of the true significance of cinder concrete proper, particularly as it relates to New York experience. From 1896, when this type of construction first received recognition in New York City, after full-size fire and load test investigations by the New York Bureau of Buildings, until 1915, the industry has been continually experimenting with various proportions of sand and cinders and of cinders alone in combination with portland cement. Much of this research was done prior to the publication of the "water-cement ratio" and the "fineness-modulus" theories. Nevertheless the development of the 1:2:5 sand-cinder mix is not an arbitrary choice. It was arrived at by the same methods as the 1:7 and the 1:9 cinder mixes of the block industry; and proved its effectiveness as a fire-resistive concrete of sufficient strength and density combined with relatively light weight. It was found that the run of commercial cinders resulting from forced draft burning is generally deficient in fine material and that the cinder fines are of such nature as to resist combination with the cement and water. In a batch, the mixture results in a harsh, unworkable concrete. Some of the earlier approvals granted by the City of New York included mixtures of 1 cement to 5 or 7 cinders. In general, however, these harsh mixes have been abandoned. Moreover, sanding not only gives a more workable mixture but actually shows greater economy in cement yield. Another difficulty which developed with the use of a graded cinder aggregate is the tendency toward segregation of fine and coarse aggregate in transportation and handling. The addition of a silica sand to the mixture was found to be a practical solution of the problem.

Though the introduction of sand to commercial cinders increases the weight of the concrete, it is generally accompanied by increased strength. The author has cited some figures to prove the contrary. The conclusion drawn from the investigations at Columbia University for the New York Bureau of Buildings as reported in Vol. LXXIX, Transactions of the American Society of Civil Engineers was that 800 lbs. per sq. in. is the

minimum compressive strength desirable or necessary to develop the safe load capacities of floor slabs as determined by the empirical formulas developed from these tests. The investigators were not aiming at 800 lbs. per sq. in., but determined that this strength could be practically attained and was sufficient for the purpose of providing an economical, fire-resistive floor filling between structural beams and girders, with an economical cement factor. Higher strengths can be and have been secured by modern control methods; but are economically impractical in application to the short span floor construction. With the modern demand for minimum story heights and maximum number of stories in a given height, there is a tendency toward the use of longer span floor constructions typified by the two-way reinforced slab supported on four edges. The writer believes that the use of a light-weight controlled cinder concrete mixture will be highly desirable for this use, and that further research for this purpose may be required along the lines of controlling the cinder aggregate.

To demonstrate the quality of concrete possible with New York materials and methods, the following observations might be cited. The cinder concrete made on the unsupervised job, as determined by job cylinders of presumably a 1:2:5 mix averaged from 500 to 1000 lbs. per sq. in. compressive strength. Job cylinders made from the same component materials over which the investigators had absolute control of the proportions, produced strengths of 1100 to 1500 lbs. per sq. in. Laboratory cylinders of the same component materials in a 1:2:5 mixture gave strengths as high as 1700 lbs. per sq. in. at 28 days. These data are reported in the paper above referred to. More recent tests on cylinders from jobs in New York City show strengths of 1:2:5 sand-cinder concretes as high as 1300 to 1500 lbs. per sq. in. and the same component materials mixed under laboratory conditions developed strengths as high as 1750 lbs. per sq. in. These values compare very favorably with Dr. Humphrey's results of 1000 to 1200 lbs. per sq. in. for the 1:2:5 and 1500 to 1900 lbs. per sq. in. for the 1:2:4 sand cinder concretes. These too are laboratory results. The amount of mixing water is not reported in the paper for the New York material, but this in general, to the writer's recollection, averaged 12 to 15 per cent by weight of the combined dry mixture or approximately 10 to 12 gals. of

water per sack of cement. Water content was carefully noted in the laboratory specimens and the records probably still exist although not available at the moment of writing. The field specimens were not under the control of the investigators and were purposely selected to represent commercial practice.

By reason of the nature of the poured-in-place construction, any cracks that develop from the volume changes take place along the natural cleavage planes furnished by the enclosing steel frame. The concrete being poured with sufficient water content to give flowability and to coat the embedded reinforcement with cement mortar hydrates sufficiently in the first few months of its life. Thereafter there is little moisture change on the interior of the average occupied building, and consequently little further volumetric change. This is not true of the factory-made, dry cinder unit which is cellular in nature, highly fire-resistant, but purposely manufactured to give a rough surface texture for architectural and ornamental treatments, if in any way subjected to the elements.

The need of a smooth-faced sand in the plastic poured-in-place cinder concrete is quite evident to anyone who has had experience with this aggregate. If more cinder fines are added to the mixture to produce workability, the cement factor becomes uneconomical. In the cinder mix without sand, the water fills the voids of the coarse aggregate and carries some of the cement with it into these cores. The cinder fines do not combine with the cement to form a plastic mortar. The general appearance of the mixture is a mass of coarse unattached particles which are harsh in appearance as well as in working qualities. When the mix is allowed to stand, the water drains out of the cell-like structure of the aggregate and is lost. The addition of the smooth-surfaced particles of sand forms with the cement a plastic matrix which coats and binds the coarse aggregate together. Unlike the cinder fines, the sand tends to hold the water in suspension where it can combine freely with the cement particles.

The writer agrees with Mr. Christensen that there is an extended field for the use of properly controlled sand-cinder concrete, such as for long-span floor constructions. For such usage, further investigation is required to determine the correct water-cement ratio and grading of fine and coarse particles. Cinder

aggregate is generally cellular rather than porous in character and requires a cementitious paste to close these voids and bind the particles together. An excess of cement would accomplish this purpose in a cinder mixture, but not as economically as additional sand in a sand-cinder mixture. The former is a lighter-weight product but lacks the properties of a good concrete.

In connection with the development of a controlled aggregate for light-weight concrete, made by submitting ordinary house ash to a smelting process whereby most of the combustible constituents of the ash are consumed and the non-combustibles are fused into a hard, vitreous clinker of iron-aluminum silicates, and then crushed, screened and graded to a predetermined sieve analysis, an extended investigation was made of the sizing of particles and proportions of fine to coarse aggregates required to produce a practical and workable concrete mixture. This research was done at Columbia University by John S. Peck, Testing Engineer, and the writer has requested Mr. Peck to submit a digest of his findings as a contribution to this subject. The aggregate resembles cinders in many respects, contains about 50 per cent voids, and presented similar problems in the making of concrete, in that an economical, workable concrete could not be produced without sanding the mixture.

BY JOHN SANFORD PECK*

The writer recently spent considerable time on the problem of developing a workable and commercially practical mix using a manufactured material that was projected for use as a light weight aggregate. It was in effect a synthetic cinder.

Very illuminating results developed from this work, and indicated the fallacy of a generally accepted theory. In the writer's opinion, the fineness modulus is of little use in designing concrete mixtures. It is outworn and discredited by the latest practice.

The writer found that minor variations in the grading had no effect on the strength of the concrete. A pinch more or less of one size or another represented wasted effort. The only thing that did affect the result was the relation between the volumes of coarse and fine aggregate in the mix, calling all material

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coarser than ¼ in. "coarse aggregate" and all passing ¼ in., "fine aggregate."

The condition necessary for maximum strength plus maximum workability was, that there should be present enough mortar i. e., fine aggregate and cement, to fill the voids in the coarse aggregate and a little excess to promote workability. In this statement the writer is sustained by no less an authority than Professor Talbot of the University of Illinois in his work on the mortar void theory.

The writer started with a grading that was made up in the laboratory to approximate closely Fuller's Curve of Maximum Density. Call this Grading No. 1. When this mix was tried it was obviously lacking in fines and therefore grading 1A was made up by adding fine aggregate until the ratio of fine to coarse was 1 to 2 by volume. Grading 1B was also made with a fine-coarse ratio of 1 to 1.5. A third grading was made in which the fine-coarse ratio was 1 to 1. Using a real mix that gave a constant cement factor of 5½ bags of cement per cubic yard of concrete, and with all other factors equal, these gradings gave the following results:

Grading	Fine-Coarse	Compressive Strength at 28 Days Lbs. per sq. in.
1		1523
1A	1 to 2	1852
1B	1 to 1.5	1957
1C	1 to 1	1883

These results show that the mix most efficient in strength was one with the fine-coarse ratio of 1 to 1.5. In the writer's opinion, the slump cone does not show workability and to date no acceptable quantitative method has been developed. The workability therefore was judged, not by the slump cone but by the general characteristics of the mix, its "fatness," and the ease with which it could be rodded into the cylinder mould. Grading 1 had no workability at all. It was harsh and rebellious. Grading 1A was not much better, but grading 1B gave a near approach to plasticity and grading 1C gave a true plastic mix.

Mr. Christensen is closer to the real answer to the problem when he speaks of splitting cinders as received on a ¼ inch screen

and recombining on the basis of required strength and workability. The average run of commercial cinders as used in New York is too harsh and lacking in fines to produce either strength or workability and the use of sand to supply that lack is more economical than screening and recombining the cinders to secure the proper proportions. It is the logical conclusion, and experience has amply proved that the 1:2:5 sand-cinder mix fulfills the requirements.

Tens of millions of square feet of sand-cinder concrete construction have been installed in New York City and in no case has fire collapsed a single arch nor has failure occurred where the slab has been in place more than 30 days. To crush or screen the cinders which form such an important part of this construction would be to load up an efficient construction with an expense that would not be justified.

BY E. H. PRAEGER*

All of the many different building materials now available may be used to advantage in the proper place. The author of this report, although actively connected with the production of cinder concrete materials, calls attention to the fact that, while cinder concrete has many virtues, there are conditions under which it may not be suitable.

Cinder concrete has been used as a building material for some thirty odd years. It was developed as a fire-proof floor system and approved in New York City after a series of tests. During the first 10 or 15 years, many tests were made and there was considerable discussion both by those who favored and those who opposed cinder concrete as a floor construction. In the last 10 or 15 years, however, the subject seems to have been forgotten.

The pre-cast cinder concrete unit is of more recent origin, it having been developed as a block for partitions and load-bearing walls for light construction. In some localities these blocks have been used for exterior walls with only a coat of cement paint to act as a water-proofing agent and some very pleasing architectural results have been obtained. Where storm conditions are more severe, this method is not suitable and it becomes necessary to use stucco, brick or stone as a facing material. Recently pre-

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cast cinder concrete lintels have been developed in connection with the use of cinder concrete block construction. A still further development is the use of cinder concrete for pre-cast floor joists. These joists are installed in a manner similar to wood joist construction. The investigations to date seem to warrant a future for this type of construction for light floor loads in places where cinders are available.

The author has covered the subject of cinder-concrete, especially in regard to pre-cast units, in an extremely comprehensive manner. There has been considerable prejudice against cinder concrete at different times, but most of this is probably due to ignorance, and the publication of accurate scientific data and results of tests should help to clarify the existing hazy atmosphere. Recently, the writer was called in by a newly elected mayor at a formal meeting of the school board of a suburban town to advise as to tearing down and rebuilding a new one-story school, then well under construction, because the contractor had used cinder blocks in backing up his brick facing. There had been so many rumors concerning this material that the parents were extremely skeptical about permitting their children to attend school if this alarming condition were not rectified.

It is not surprising, however, that erroneous impressions may exist among laymen, when it is realized that the engineer, even now, has comparatively few data in regard to cinder concrete as used in floor construction. Furthermore, this condition is not helped by those who specialize in other somewhat similar materials and who emphasize the faults in cinder concrete, many of which may or may not exist. Some of the criticisms of cinder concrete are admittedly justified, others are not. This material was developed as a substitute for brick, terra-cotta and stone concrete floors and the questions of fire resisting qualities, corrosive effect on metal (structural steel, reinforcing steel and pipes), permanency, strength, weight, economy, design, manufacture and installation have been discussed at length.

In regard to fire-resisting qualities, laboratory and actual fire tests have demonstrated that a well mixed cinder concrete is at least the equal of any fire-proofing material which may be used in a similar manner.

In regard to corrosion of steel, investigations have proved that the greatest part of the criticism from this source is unjustified. Evidences of corrosion have been noted, but in most cases the cause has been traced to faulty materials or workmanship. In order to obtain the most satisfactory results with cinder concrete, the design, materials and workmanship must be of a high order. It must be admitted that the above requirements do not always exist and our best efforts should be exerted towards improving this condition. The investigations of the author should dispel the overworked and unwarranted fear of the effects of sulphur that may be contained in the cinders. The following paragraph from an article in the *Builder* (England) under date of September 12, 1930, illustrates a very recent English viewpoint.

> Clinker concrete should never be used to cover steel nor be reinforced, on account of its porosity and probable presence of sulphur in the clinkers.

In cinder fill pipes and conduits should be covered with cement mortar before the fill is placed, but in this case the conduits need not be of galvanized metal except in special instances.

As to permanence, we can only state that many square feet have been in place for over a quarter of a century and give no evidence of deterioration. There should be many excellent opportunities for testing actual installations to destruction, as several fire-proof buildings with cinder concrete floors have been demolished and certainly many more will come down in the near future, in order to provide room for more up-to-date buildings. Extensive tests of old slabs in these buildings would give a truer indication of the characteristics of this construction than isolated laboratory tests.

The compressive strength of a uniform run of cinders can be determined, but this may not be relied upon as an accurate comparison of what may be expected in the floor slab as installed in the building, as results of field tests have varied by more than 100 per cent as compared to laboratory tests. The fact that cinder concrete is not as strong and as dense as stone concrete may be an advantage in light manufacturing buildings, the various sections of which may be subject to local vibrations. Such vibrations will be absorbed within a small area and not transmitted throughout the entire structure as would be the case with stone concrete.

As cinder concrete weighs only about 70 per cent as much as stone concrete naturally this fact has a considerable bearing on the tonnage of structural steel as well as on the cost of foundations of a tall building.

As to economy of the structure compared to other systems, this is dependent upon the location of the job and the conditions of the market. When structural steel is low and where cinders are available, it is difficult for any other fire proof system to compete with cinder concrete floors. In the middle west reinforced concrete framed buildings, of the apartment house type, are considerably cheaper than steel-framed buildings with concrete floors, but in New York City steel framed buildings with cinder concrete floors are more economical than any other type of fire proof construction.

There are probably more unknown factors in regard to the design of so simple and yet so important a member as cinder concrete slabs than exist in the design of some of the most complicated engineering structural members. In some localities cinder concrete is designed in accordance with the usual theory of flexure, using about one-half the stress allowed for stone concrete and several other assumed values of constants. In New York City no mention is made of the stress in the cinder concrete but empirical formulae are used to determine the area of reinforcing for different loads, spans and conditions of restraint. These formulae are based upon the results of laboratory tests, which tests are often quite unlike actual job conditions. Whether the concrete acts as a slab, as a flat arch, as a filler between the reinforcing which is in suspension, as a combination of any or all of the above, or whether or not it acts at all, are questions that have not been satisfactorily answered after many years of extensive use of this material.

The fact remains that in different cities entirely different requirements obtain as to the design of cinder concrete slabs. These different requirements give results which cannot be compared, the empirical formulae permitting loadings which in some cases would stress the materials beyond their ultimate strengths as determined by the usual concrete formulae. The empirical formulae were deducted from tests, and the fact that other than

slab action exists is self evident. However, the question of design should receive further study and tests should be continued until we arrive at results which will check with some theory or combination of theories. At this time, to cite only one example, New York City would permit a superimposed load of 400 lbs. per sq. ft. on a 4 in. cinder concrete slab on an 8 ft. span provided 0.36 sq. in. of reinforcing mesh were used per foot of width. For this span and equivalent total load, Los Angeles would require a 6 in. stone concrete slab with 0.46 sq. in. of reinforcing rods. According to the usual concrete formula, the compressive stress in a 4 in. stone concrete slab with this span and load would be 1200 lbs. per square inch. It probably is fortunate that floors are seldom loaded to their full design loadings and it is my opinion that the present empirical formulae are too optimistic and should be made more conservative.

Attention may here be invited to the fact that cinder concrete slabs, which also encase the supporting steel beams in the usual manner, materially increase the strength of these supporting beams and girders. A very complete series of laboratory tests has recently been conducted under the auspices of the Western Society of Engineers, in which cinder concrete and gravel concrete slabs, supported by steel beams (some of which were encased in concrete and others were not) were loaded with several different loadings and readings were taken and recorded. The stresses and deflections of the supporting steel beams and girders were measured by a strain gage and a deflectometer. These quantities were then computed by the usual formulae and the results tabulated as percentages of measured to computed stresses and deflections. These comparisons clearly show the assistance rendered to the structural steel by the concrete fireproofing, especially at design loads. The concrete acts as a stem of a T beam, thus increasing the compression area and raising the neutral axis. Whether or not the same results may be expected with average job conditions is a debatable question. However, as a result of these tests the committee unanimously recommended 24,000 pounds per square inch as the allowable unit stress in structural steel I beams up to 24 in. deep when properly encased in either stone or cinder concrete. Incidentally, the committee recom-

mends the adoption of the empirical formulae for cinder concrete now in use in New York City.

The manufacture and installation of cinder concrete is probably even more important than the question of design. We are living in an age of specialization and while this often proves an advantage, it sometimes has the opposite effect. Cinder concrete is a separate trade and this work is rarely performed by the general contractor. Although the materials are defined in the specifications and the proportions, time of mixing, method of placing, etc., clearly and carefully covered, these requirements are seldom adhered to rigidly. Recently, a reliable cinder concrete contractor, on signing a contract with a general contractor from a near-by city, stipulated that his quotation was based upon his being permitted to install his work without interference by the general contractor. This sub-contractor had no intention of purposely "skinning" the job but he knew that the general contractor was familiar with stone concrete and unfamiliar with cinder concrete installation and rather expected him to be somewhat shocked at the comparison. There seems to be an unwritten law that as far as cinder concrete is concerned, almost anything will do. The specifications for cinders, as proposed by the author, are a step in the right direction, but specifications alone do not insure results. When a load of cinders is dumped in the hopper to be used a few minutes later, it is too late to check up the specifications. The answer of the material man is that they are the best he can get right now, and it is either a case of take what you can get or nothing. There is plenty of room for improvement on the job in regard to careful proportioning of the materials, time of mixing, quantity of water, method of placing and other job conditions. The material is generally mixed in the basement in a continuous mixer and neglected from the time it arrives as raw materials until the forms are stripped. If the designated size of mesh has been stretched over the steel beams, it does not seem to make much difference what location in the thickness of the slab the mesh is placed. Failures have been few but there have been some and it is believed that a more rigid control of job conditions will materially improve the finished product at a very small additional expense.

The possibilities of painting and decorating cinder concrete

directly, thereby saving the cost of lathing and plastering has been given some study and there have been several jobs where the results have been quite satisfactory when sufficient precautionary methods have been taken.

The field tests suggested by the author should be included in the specifications of all important jobs and test cylinders should be taken frequently if for no other reason than the psychological effect on the workmen. Exact proportions of mix, preferably not less than 1:2:4, should be insisted upon. Sieve analyses can readily be made as hand sieves are inexpensive and accurate enough for job control. The quality and uniformity of the cinders can be guaranteed if the engineers insist.

A step in the right direction would be taken if some of the factory methods of manufacturing cinder concrete units were transposed to the job.

BY EINAR CHRISTENSEN

Mr. Strehan opens his discussion with the statement that "the cinder concrete in the factory-made cinder unit of today is not well-made cinder concrete at all." This is decidedly contrary to the writer's experience, and does not appear to be in any way proven by the arguments advanced by Mr. Strehan.

It should be borne in mind that while the mixture used in the making of cinder units is not plastic, it is molded under high pressure. The process is therefore entirely different from that of placing concrete on the job, and the question of consistency cannot be considered in the same light.

As a matter of fact, factory-made 1:7 cinder concrete contains almost as much water per sack of cement as the 1:2:5 sand-cinder concrete discussed by Mr. Strehan. This water does not "drain off rapidly," as Mr. Strehan claims—it is held for a long time by the cellular concrete and is available for hydration of the cement to the same degree as in a sand-cinder concrete.

It is unjust to say that the matter of mixing water is generally neglected in cinder unit plants. The mixer man quickly learns to judge his mix with a fair degree of accuracy, and the figures of 4 to 5 lbs. of water per cubic foot of aggregate, suggested by Mr. Strehan, do not in any way represent mixtures actually used. The figures given by the writer, of approximately 1.5 gallons (12 lbs.) per cubic foot, are typical.

Mr. Strehan assumes that this "dry mixed" cinder concrete causes trouble in the wall. He says that "the high expansional movement of cinder concrete normally associated with its early life is very likely to be prolonged to the masonry in the wall because of the slow hydration of an all-cinder concrete mixture." The fact remains, however, that volume change of cinder concrete masonry, if occurring at all, is a contractional movement similar to that of other concretes, accompanying further hydration of the cement and the drying of the masonry. As shown by the tests quoted in the report, such shrinkage movement is avoided when the concrete units are properly aged and dry.

The writer will be interested in knowing the facts that justify the statement of "lack of durability or permanence in the structure as well as the corrosion of embedded steel occasionally demonstrated by the rusting of nails driven into the masonry." It is contrary to the ten-years experience of the writer in the cinder unit industry. The product has enough history back of it to show its permanence—and the almost universal nailing of trim and grounds to cinder concrete masonry affords ample opportunity to observe the conditions of the nails. That nails driven into the outside of walls may rust, is natural when they are exposed to rain and not sufficiently protected by the surrounding concrete.

To Mr. Strehan, sand is a necessary constituent. That it is frequently desirable in order to produce workability in job-placed concrete is true—that it does not of necessity improve the concrete is shown in the paper. And matters are quite different with cinder concrete building units, molded under heavy pressure.

Mr. Strehan does not think much of the "cinder fines." Nevertheless, his own tests (quoted in the report) show a decrease in strength of sand-cinder concrete when the fines were screened out. Examination of the interior of cinder concrete (molded under pressure) shows a homogeneous concrete—cellular, but nothing like the "mass of coarse unattached particles" pictured by Mr. Strehan.

The supplement by Mr. Peck contains some interesting information regarding the effect of grading. Apparently, the fine-coarse ratios reported have reference to a straight cinder aggregate, indicating what may be accomplished without the addition

of sand. That the latter becomes important where the cinders are lacking in fines, appears obvious. But the self-contained cinder aggregate, reported to split 50–50 on ¼ in., is stated to produce a "true plastic mix."

Mr. Strehan's long experience with sand-cinder concrete in New York adds valuable data on the nature of the New York material. That this "sand-cinder concrete" does not behave exactly like the "cinder concrete" used in machine-made units, is one reason why the writer suggested the two names to differentiate the different materials.

To sum up: It appears to the writer that Mr. Strehan fails to appreciate the fact that factory-made cinder concrete is different from job-made sand-cinder concrete, not merely in having a straight cinder aggregate (graded from coarse to fine) and in having a different consistency, but also in that it is molded under pressure and cured under conditions where the aging can be controlled. Actual performance and accelerated laboratory tests have shown that such factory-made cinder concrete is permanent and offers full protection for any steel properly embedded therein.

The writer has little to add to the interesting discussion by Mr. Praeger. While the matter of design of reinforced cinder concrete was considered to be outside the scope of the report, it is nevertheless an important one as pointed out in this discussion, and it is to be hoped that further work will be done to develop more accurate and uniform specifications.

Readers are referred to the JOURNAL *for December 1931 (Vol. 28), for further disussion. Such discussion should reach the Secretary by Nov. 1.*

A Discussion of a Paper by Raymond E. Davis and Harmer E. Davis:

"Flow of Concrete under the Action of Sustained Loads"*

BY G. A. MANEY† AND M. B. GAMET‡

The well established fact that plastic flow is increased 200 to 300 per cent or more by the drying out of a specimen, which, is identical in all details of mix and strength, leads us to look to shrinkage for an explanation. A paper is being prepared describing the results of a series of experiments at Northwestern University which gives an explanation not only of this phenomenon, but of the decrease in the modulus of elasticity of dry specimens which are identical in strength and mix with specimens tested wet.

A series of pairs of 8 x 32-in. cylinders, one hollow with a ¾-in. shell and the other solid were investigated for shrinkage. 20 in. longitudinal gauge lines were read on the surface of both specimens, and a special 32-in. instrument was designed to read between end points, as shown in Figs. (4) and (5) and was used to get the variation of shrinkage at the several points indicated in the interior of the specimen.

Our best information at present would indicate that the surface shrinkage of the solid specimen will eventually approach closely that of the hollow one, but a tremendous lag is seen in Fig. 3 at all intermediate ages. As this paper is in the nature of a progress report, only 20-day values are available.

Fig. 6 shows the comparative deformations of solid and hollow specimens at age 20 days, and Fig. 7 indicates approximately the

*A. C. I. Journal, March, 1931; A. C. I. *Proceedings*, Vol. 27, p. 837. Submitted as a part of the work of Committee 109, Summary of Plastic Flow, Raymond E. Davis, author-chairman.
†Professor of Structural Engineering, Northwestern University, Evanston, Ill.
‡Instructor in Civil Engineering, Northwestern University, Evanston, Ill.

stress distribution which must exist under *no load*. The surface tension f_t is measured by the difference between the unrestrained surface shrinkage e_c, as obtained from the shell, and e'_c which is a restrained surface deformation, greatly reduced below e_c, because of the very slow shrinkage of the interior of the solid specimen. Figs. 8 and 9 show the difference between probable and commonly assumed stress distribution.

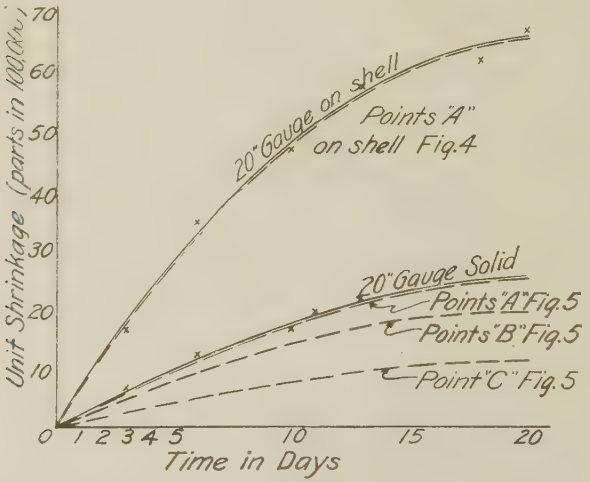


FIG. 3—CURVES SHOWING RELATION BETWEEN TIME AND SHRINKAGE IN SOLID AND HOLLOW 8 IN. X 32 IN. CYLINDERS FOR 1:1:1 MIX (BY WEIGHT) WITH APPROXIMATE 8 IN. SLUMP

If we assume, as is closely true for wet specimens which eliminate the shrinkage problem, that flow is proportional to unit stress, this distribution of stress explains all of the high flow values obtained in the dry specimens discussed in this report. Under low loads the actual stress may be 3 to 6 times greater than the average stress, and only (in the case of the specimens discussed here) at a deformation difference of .00040 which represents 1600 lb. per sq. in. at age 20 days (using $E_c = 4{,}000{,}000$) would this ratio come down even to a 2 to 1 ratio. It is doubtful in a solid specimen of appreciable size whether the interior shrinkage ever reaches the surface value.

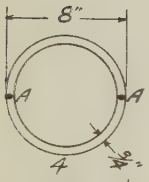


FIG. 4—PLAN OF END OF HOLLOW SPECIMEN

FIG. 5—PLAN OF END OF SOLID SPECIMEN SHOWING LOCATION OF PLUGS FOR READINGS WITH SPECIAL MICROMETER INSTRUMENTS

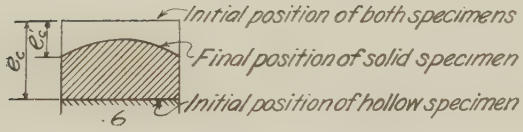


FIG. 6—COMPARATIVE UNIT DEFORMATIONS OF SHRINKAGE ON HOLLOW AND SOLID SPECIMENS

FIG. 7—APPROXIMATE STRESS DISTRIBUTION UNDER NO LOAD



FIG. 8—STRESS DISTRIBUTION UNDER LOAD AT TIME WHEN OUTSIDE TENSION IS JUST ELIMINATED

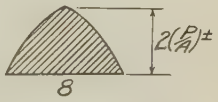
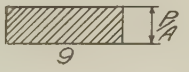


FIG. 9—COMMONLY ASSUMED UNIFORM STRESS DISTRIBUTION



We also wish to submit Figs. 1a to 1d and Fig. 2 (See page 1299), which are practically self explanatory, and which bring out forcibly the fact that a companion specimen with the same steel as a loaded specimen has no bearing on the problem of how the total time increment is divided between shrinkage and plastic flow. In general, when the proper control specimen (which is always plain concrete for a column with vertical steel) is used, shrinkage will account for all of the stress due to time, if no load is applied directly to the steel, but gets to it only through bond as is universally the case in practice. In the specimens of the paper discussed, since load is transmitted directly to the ends of the steel bars, any plastic flow deformations in the concrete cannot take place without carrying the steel along. In practice this is not the case, and so far the specimens tested in the Northwestern University laboratory indicate that flow can take place in concrete around the bars without in any way affecting their stress.

BY J. R. SHANK*

The writers of this paper are to be commended for the easily readable presentation of this work. The writer of this discussion, who had no part in the work of this paper, has some data which he has obtained under different conditions of materials and procedure which he would like to present. Four curve sheets and a sketch are presented bearing on the effect of mineral character of aggregates, on the effect of the amount of load, and on the creep or flow of reinforcing bars in compression splices. These are the results of bachelors' theses carried out at the Ohio State University under the close supervision of the writer.

Fig. 1 shows the results of tests of concretes made up of aggregates of two widely different characters, crushed limestone and silica gravel. The same material was used for both coarse and fine in all specimens. These specimens were made up and tested in a manner similar to that described by the writer in a discussion A. C. I. *Proceedings*, 1928, of a paper by Raymond E. Davis on this same subject. The limestone used was a local product known as Columbus limestone. It is a rather hard fossil-bearing limestone used for concrete and to some extent as a building stone. A notable example of its use as a concrete aggregate is in the Ohio Stadium and as a building stone in the State Capitol at Columbus.

*Professor, Dept. of Civil Engineering, Ohio State University. Critic member Committee 109.

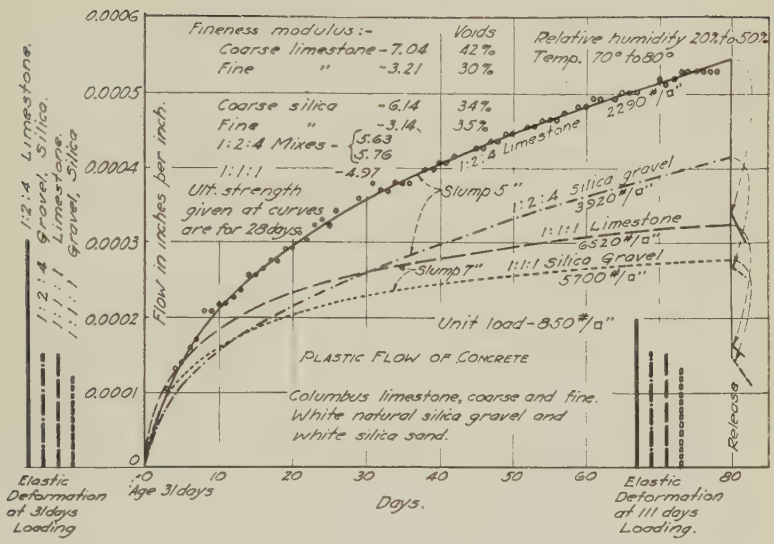


FIG. 1—CURVES SHOWING PLASTIC FLOW OF CONCRETE

It is of a yellowish white color and it breaks in crushing to particles of a generally regular polyhedral shape. There is some tendency to break into somewhat elongated pieces, but none into flat plate-like shapes. The fine limestone aggregate had the fine flour washed out. In this shape it is a commercial product sold for concrete construction. The silica material was obtained from a gravel bank near Akron, Ohio. It has nearly white translucent smooth particles of quartz both coarse and fine.

The investigation was made on two mixes, one a rich mix rather wet and the other a more common mix. The conditions of fineness modulus, consistency and curing conditions were the same in the specimens of each mix. Considering the richer mix, it will be noticed that the amount of flow at 80 days is very nearly inversely proportional to the strengths obtained. This is contrary to the general expectations. The stronger concretes usually flow less. The instantaneous deformations and recoveries appear to be very nearly the same though one occurred at 31 days and the other at 111 days. The leaner mixes show limestone with the greater flow and the lower strength, and the difference is more marked than for the rich mixes.

These specimens were kept during the loading time in ordinary room temperatures and humidities after having been cured in damp sand at about 75 deg. F. room temperature for 31 days. The time of the year was winter and spring. The room was steam heated and the temperature ranged between 70 and 80 deg. F. The relative humidity varied from 20 to 50 per cent which is usual for the interiors of heated buildings for this time of the year where no effort at humidifying the air is made. The changes due to temperature and humidity variations are eliminated from these curves by correcting each reading by a deformation observed on a similar specimen from the same batch cured and stored at all times with the specimen in the test, and treated in all cases the same excepting that it was not under external loading. All four loaded specimens with their respective idlers were kept in the same room. These curves are therefore comparable with each other and can be compared with other curves for specimens under about the same conditions of temperature and humidity. They certainly represent the condition of concretes inside of buildings.

In the cases of both mixes whether stronger or weaker the limestone concrete showed more flow than the silica gravel concrete. This is contrary to the results shown on Fig. 6 of the report. The non-agreement of the results from these two sources indicates that there is considerably more to be learned about the effect of the mineral character of aggregates on flow. As a part of this work an attempt was made to look into the plastic flow properties of aggregate materials themselves. Specimens of three common building stones were secured and put through the same tests as were the concretes. Fig. 2 shows the results. The Columbus limestone came from the same quarry from which the limestone aggregate was taken. The Indiana limestone was furnished by the Indiana Limestone Co., Bedford, Indiana, from quarries of the well known oölitic Bedford or Indiana limestone. The Amherst Sandstone was furnished by the Ohio Cut Stone Co., Cleveland, Ohio. This stone is a fine grained light buff colored silica-bound sandstone. It has a high rating as a building stone. Two specimens were tested, one along the grain and the other across the grain. The specimens were all loaded and unloaded once or twice

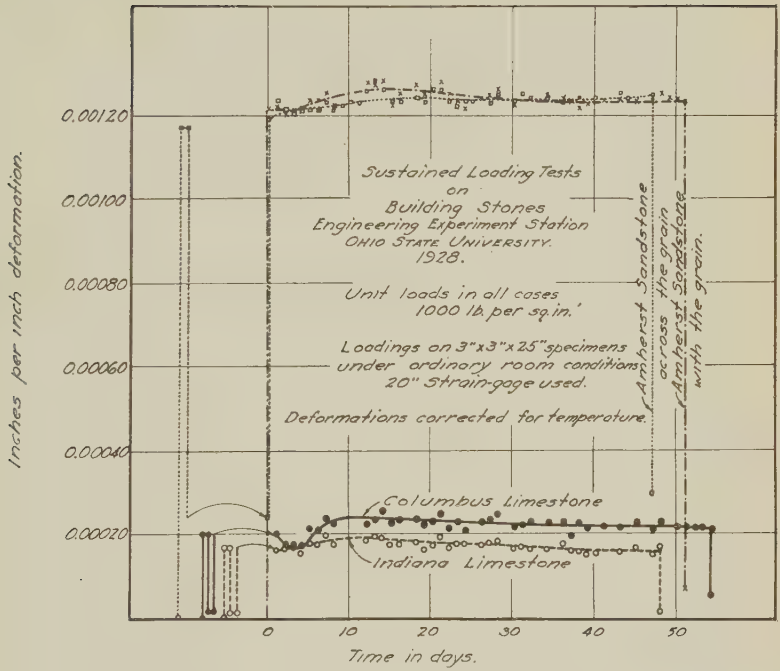


FIG. 2—SUSTAINED LOADING TESTS ON BUILDING STONES

before they were given the sustained loading. Fig. 2 shows this to the left of the sustained loading graphs.

It will be noticed immediately that there is no appreciable plastic flow in these building stones. The Amherst stones seemed to show some in the first few days which, however, was hardly apparent at the end of the 50 days. It is interesting to note the large differences in the elastic properties of these stones and that Fig. 6 of the report shows sandstone as an aggregate to have the greatest flow while Fig. 2 of the discussion shows the Amherst sandstone to have the low modulus of elasticity. It is regretted that Fig. 1 shows no concrete made of Amherst sandstone as an aggregate. These few tests indicate that the plastic flow in concrete is not due to any large flow in the aggregate particles themselves. This leaves the shapes of the particles and the surface conditions to be considered. The amount of surface and the

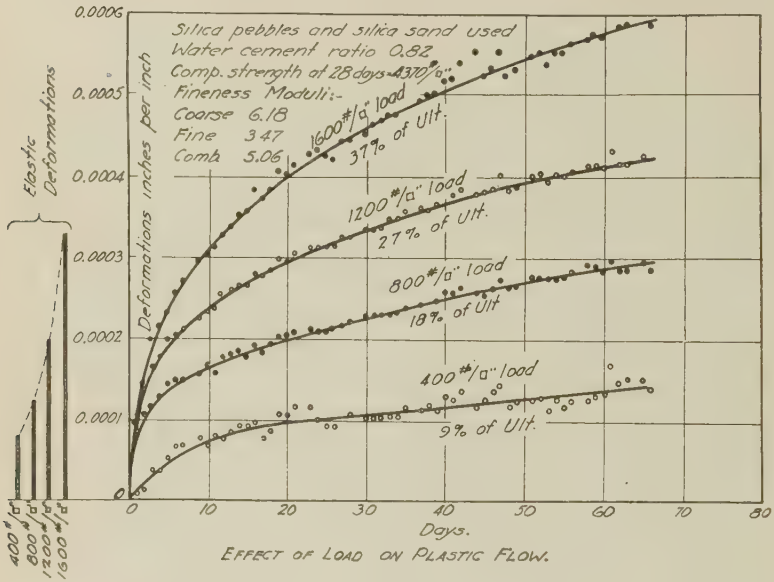


FIG. 3—EFFECT OF LOAD ON PLASTIC FLOW

character of the bonding of the cement to the surface of the aggregate particles are factors to be considered. It appears that other classifications than those given in this report will have to be made before this matter can be finally presented.

Fig. 3 shows additional data on the effect of stress upon air stored specimens loaded at 28 days. This corresponds with Fig. 10 of the report and shows similar results. The discussion shows a stronger concrete in which higher unit stresses were used. The loading period was very much shorter. Here again the conditions were those of a building interior in the winter time. The results from these two sources are in close agreement. Technical Paper No. 12 on building research of the Department of Scientific and Industrial Research of Great Britain by W. H. Glanville, Fig. 10a is also in agreement. Similar results from these three independent sources might warrant the conclusion that in general the flow is proportional to the stress within ordinary working load limits. What occurs for stresses nearer the ultimate load remains yet to be investigated.

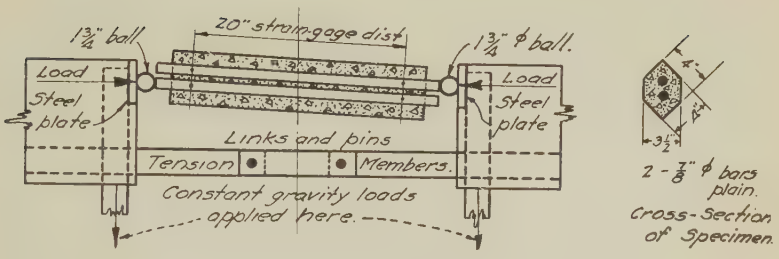


FIG. 4—SKETCH SHOWING SPECIMEN AND METHOD OF LOADING

Figs. 4 and 5 illustrate the results of an attempt to find out whether there is an appreciable bond flow or shear flow in the splices of bars in compression members such as building columns. Fig. 4 shows part of the apparatus used for the flow tests described in this discussion carrying a specimen for this particular investigation. As the load is a gravity load there can be little or no variation in the load. Specimens were made with ⅞-in. plain round bars close, ½ in. clear between, 1 in. clear between and 1½ in. clear between. They were made in pairs and cured in damp sand for 28 days at which time one of each was set up in one of the loading devices. A load of 12,500 lb. was applied to each. The specimen whose bars were 1½-in. clear between failed by diagonal tension. Another pair of specimens of similar concrete was made up, cured and loaded at 28 days with half the load. Strain-gage readings 20-in. long were taken on two sides of the specimens, two on each bar and four from bar to bar. From these data the horizontal movements of the points at one end on one bar was computed with respect to the points at the same end on the other bar. The results for the two ends of the specimen averaged gave the horizontal movement of one bar with respect to the other. These results are shown on Fig. 5 together with the flow of plain concrete of the same batch and of the same cross-section. The unit stress on this plain concrete was 915 lb. per sq. in. For the specimen having bars 1½-in. apart the deformation data were doubled before plotting so as to make the results comparable, assuming that the flow was proportional to the load. To

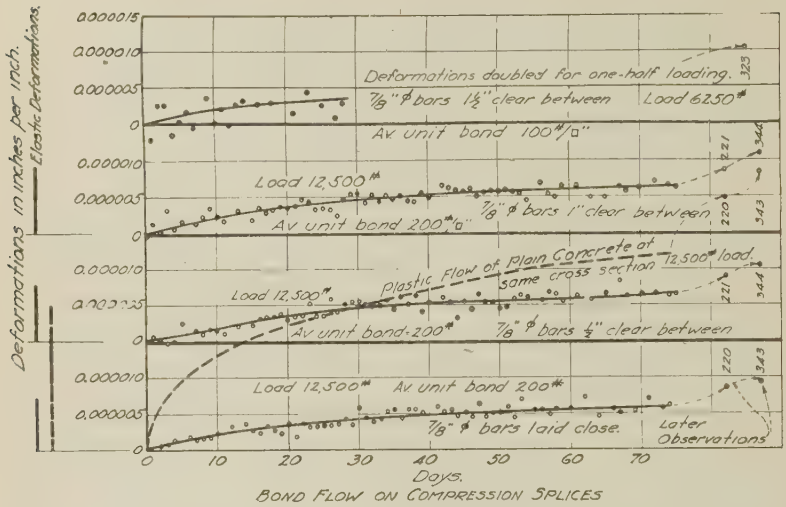


FIG. 5—BOND FLOW ON COMPRESSION SPLICES. MIX 1:2.24:2.52 SAND AND GRAVEL W/C = 0.80, SLUMP 3 IN., COMPRESSIVE STRENGTH 4200 LB. PER SQ. IN.

the data of the original thesis were added deformations at later dates which are shown as separate points to the right of Fig. 5.

It is clear that there is a movement of the bars with respect to each other and that this movement must be in the region close to the bars since the distance between the bars has very little effect. We might say that we have "bond flow," but little or no "shear flow" even though the diagonal tension arising from the shear was near the ultimate as was evidenced by the failure of the specimen having bars 1½-in. apart. At the latest date shown the movement for all specimens did not vary from 0.00001 in. per in. by more than 8 per cent. The doubled value for the specimen having bars far apart was within this limit at 297 days. At 74 days loadings the plain concrete flow was 4.3 times the average bond flow, at 220 days, 4.1 times, and at 343 days 3.8 times. This situation must make considerable difference in the distribution of stresses in tall building columns where bar splices occur at every story height. The average unit stress in the steel is, therefore, not as much as the total shortening of the column would indicate. There would also be variations in the unit stresses along the

lengths of individual bars, the greatest being at the middle of the length and usually at the mid-story heights. The carry into the column of bending from floor beam deflections would be greater into the column below the floor than above, since splices are usually made just above the floor. The point of inflections would then tend to be below the mid-story height or where full elastic action on the column would indicate.

BY LARS JORGENSEN*

The paper "Flow of Concrete under Sustained Stress" contains information of a very important nature.

It is still customary when calculating stresses in concrete structures to use the E corresponding to the instantaneous deformation, where—as is shown in this paper, the flow for sustained load greatly influences the working E.

These tests, the writer understands, were started with the object of finding the elastic property of the concrete, which went into the Stevenson Creek dam.

On dam construction an additional factor enters into the problem, namely the watersoaking effect. The results of these tests show quite conclusively that the E for wet concrete is much higher than the E for dry concrete. This is especially of considerable importance in the calculation of dam structures, where part of the dam body is wet and part of it dry, which is always the case when the structure is loaded.

For dam design another property peculiar to concrete is of great importance and that is the watersoaking effect or rather its distribution throughout the section. If the authors would extend their investigation to also include this item, they would have the subject covered rather completely.

The distribution of the watersoaking effect is perhaps quite a difficult subject to investigate, but its importance will justify any earnest attempt to surmount a great deal of difficulty.

For ordinary building design the result of the authors tests points towards the necessity of making two sets of stress calculations, one set using the ordinary or instantaneous E and one set using what the authors call sustained modulus of resistance for concrete under load (for say at least 2 years).

*Consulting Hydro-Electric Engineer, San Francisco.

For dam design the instantaneous E would hardly ever be considered, since the load on such structures comes on gradually and goes off gradually. Take a straight gravity dam for instance,the reservoir being first empty gradually fills up; the stress along the upstream face being first a maximum and later diminishes to nearly zero, while the stress along the down stream face gradually increases from a moderate amount to the maximum allowable compression with reservoir full.

Had the reservoir been empty long enough to produce flow action in the concrete along the upstream face, this face would probably have deflected in an upstream direction. With reservoir full the E for sustained modulus of resistance would have to be used, when calculating the deflection in a downstream direction especially after the reservoir has been full for some time.

The calculated deflection using the sustained modulus of resistance instead of the instantaneous E would be nearly double. To this deflection would have to be added a certain amount due to the watersoaking effect in the cantilever. The upstream face of the dam is 100 per cent wet and the downstream face is dry or nearly so on a well built dam. The upstream face therefore expands vertically relative to the downstream face, this occurrence is compensated for by additional deflection in a downstream direction of the cantilever and without setting up any additional stresses.

The stress distribution along any horizontal joint of a straight gravity dam is not necessarily changed on account of using the sustained modulus of resistance instead of the instantaneous E. This distribution will most likely be in accordance with the theory given by B. F. Jakobsen in a paper before the Am. Soc. Civil Engineers (*Proceedings* Am. Soc. Civil Engineers, September, 1930).

The much larger deflection due to the use of the sustained modulus of resistance instead of the instantaneous modulus of elasticity and the addition to this deflection due to the watersoaking effect is of no special importance for a straight gravity dam, but it is of great importance on a curved dam, because on a curved dam, there is a possibility, that the load may be carried on two systems, arch and gravity (inversely proportional to their deflections).

When calculating the deflection of an arch dam in a down stream direction, the working modulus of elasticity will be considerably higher than the sustained modulus of resistance for dry concrete, since the upstream face is wet when under stress. The portion of the system (cantilever) which might carry load by gravity action is dry where it has its maximum stresses. The wet upstream face expands along the horizontal arch and this is equivalent to an increase in E, a decrease in arch deflection and a decrease or elimination of tensile stress at the abutments.

The wet upstream face also expands vertically, this affects the cantilever as it produces a deflection in a downstream direction without setting up any stresses and therefore without carrying any load. Should the vertical cross section of the dam be such that the deflection of the cantilever due to watersoaking effect in the cantilever is equivalent to the arch deflection, assuming the arch to carry all the load, then the cantilever does not carry any load at all since the water soaking effect in the cantilever made it deflect without setting up any stresses.

If the cantilever deflection due to the watersoaking effect is less than the deflection due to full load on the arch, then of course the cantilever will take some load, since the deflection of the two systems must be the same.

Not so very much is known about the distribution of the watersoaking effect in a body of concrete wet on one face and dry on the opposite face and it is therefore not warranted to assume that the arch receives a definite support from the vertical cantilever.

As already stated, if the authors could be induced to carry on experiments on the distribution of such watersoaking effect in concrete, some valuable information undoubtedly would be forthcoming on this rather dark subject.

CONVENTION DISCUSSION

CLOYD M. CHAPMAN—I was asked the other day whether any tests were made to determine if the sustained load due to the car springs used in the testing apparatus was constant.

RAYMOND E. DAVIS—The springs have been periodically tested to determine whether or not there was any dropping off in load and so far as we have been able to determine within the precision of our measuring apparatus, certainly within 2 or 3 per cent, I

think probably closer than that, it would be within the precision of the measuring apparatus. Therefore we may say that no appreciable dropping off of load has taken place within the limit of the experiment. Any dropping off that has taken place is negligible.

Readers are referred to the JOURNAL *for Dec. 1931 (Vol. 28), for further discussion which may develop. Such discussion should reach the Secretary by Nov. 1, 1931.*

Discussion of Second Progress Report of Committee 105

"Reinforced Concrete Column Investigation"*

BY G. A. MANEY†

The undeniable evidence of this remarkable and much needed investigation is to throw into the discard the time honored assumption of "nf_c" as representing the *total* unit stress in compressive steel. Also the additional stress due to "time" which we may most easily call $e'E_s$, where e' represents the unit deformation due to time yield, becomes a correction which not only doubles but often trebles our unit stress due to instantaneous loading.

Having been in possession of this almost universally unrecognized fact for a number of years and having published several articles as a result of experimental investigation of the compressive steel of beams in our laboratory at Northwestern University, I have been impressed with the seeming indifference of the profession to these facts. For years we have been using a formula for reinforced concrete column design which has introduced this tendency to high compressive steel stresses and have calmly ignored this same phenomenon in the calculation of compressive beam steel. It should be noted however that not even a reasonably small percentage of those using the A. C. I. column formula have understood the conception. Many of those who did realize they were using a code which was based on high design units in compressive steel, were not willing to admit that even stresses approaching the elastic limit in compressive steel under design loads should hide the fact that at ultimate load all the three elements (vertical steel, spiral steel and concrete of core) will be developing as a minimum their full capacities at elastic limit stresses.

*A. C. I. Journal, Feb. and March, 1931; *Proceedings*, Vol. 27, p. 685.

†Professor Structural Engineering, Northwestern University, Evanston, Ill.

The large scale and wide scope of this investigation which goes for the first time into the much neglected field of time loadings must eliminate any doubt as to the astounding divergence between what the average designer of reinforced concrete structures thinks he is getting and what actually occurs. Code changes in *all* compressive steel stresses are inevitable. I see in the recognition of these important findings many possibilities for reduction of size and dead weight where (as is often the case) this is desirable. We begin to realize as we get into these results that from 50 to 90 per cent and (in many cases) all of the column load is carried by vertical steel alone. This leads us to think of a new type of column with greater percentages of vertical steel and much *higher elastic limits* in vertical steel.

Much question of the relative contributions of shrinkage and plastic flow to this tremendous time increase will arise. In our investigations of compressive beam steel under continued load at Northwestern University we have been able to conclude (see *Engineering and Contracting*, June 10th, 1930) that shrinkage has a controlling effect on time stresses in compressive steel and that plastic flow, although an important quantity in plain concrete (as has been shown by Davis and others) is negligible in its effect on increased steel stresses.

The control or companion specimen used to determine shrinkage under no load is the all important detail in separating the two effects.

An examination of Figs. 1*a* to 1*d* brings out clearly the fact that a control specimen such as shown in Fig. 1*b*, of plain concrete and with unrestrained shrinkage unit change of e_{sh} is the only kind which will give any information on the question of shrinkage contribution to the true increment of compressive steel stress. A companion specimen exactly the same as the one under load and having the same compressive steel (Fig. 1*a*) does not tell more than merely how much compressive stress shrinkage *alone*, without the help of load, can throw into steel. The resulting unit change e, has no bearing whatever on the stresses resulting from the column under a deformation e_3 which exceeds e_{sh}.

Fig. 1*d* shows the total stress in compressive steel under load and shrinkage only as $(nf_c + e_{sh}E_s)$, where e_{sh} can only be the unit shrinkage of the plain concrete companion specimen.

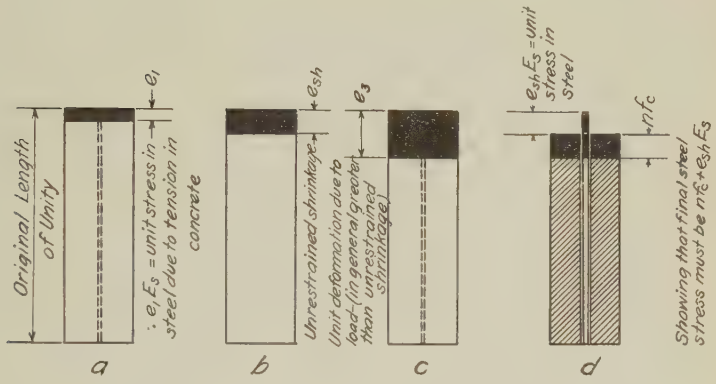


FIG. 1.

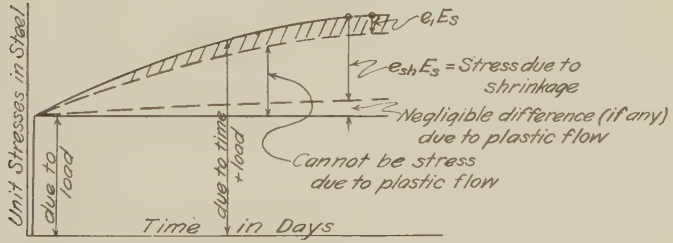


FIG. 2—CURVES ILLUSTRATING NEED OF PLAIN CONCRETE SPECIMENS FOR SHRINKAGE CONTROL

It is a notable fact that time increases in measured compressive beam steel check remarkably in an extreme range of cases with the shrinkage of plain unloaded beams of same section with no steel.

It is doubtful if plain concrete shrinkage will fail to account for all except a negligible part of total time increase in the columns of this series. In Fig. 2 is seen the difference in plastic flow effects which would result from a use of the two different kinds of control specimens.

BY WARREN RAEDER* AND HERBERT J. GILKEY†

One of the main purposes of the extensive and admirable column investigation being carried on by Committee 105 is a study of the maximum steel stresses reached in reinforced

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†Professor of Civil Engineering, University of Colorado.

columns due to live load, shrinkage, and flow, and the influence such stresses may have upon the effective strength of the column. Various investigators over the past 20 years, notably McMillan and Lagaard, have reported stresses in the steel approximating yield point values. The question naturally arises: if the reinforcing bars are stressed to the yield point, in what condition is the column to take its load?

The authors of this discussion, in an attempt to study this problem qualitatively, recently cast some compression specimens with the reinforcing bars protruding at the ends so that under test there would be no question about the bars being stressed to the yield point. This stress the bars received before the concrete ends of the specimens came into contact with the testing machine.

A description of the specimens together with the ultimate loads they took is given in Table 1. Specimens 1a to 6a inclusive were of 1:2½:4 concrete with a water-cement ratio of 1.0. The remainder of the specimens were of 1:2:2½ concrete with a water-cement ratio of 1.0 and a coarse aggregate of ⅜ in. to ½ in. pea gravel. The ½ in. square bars used extended approximately ⅛ in. at each end and the ¼ in. round bars .05 in. It will be noted that some of the bars were greased and that in specimen No. 7 the bars were greased and were flush with the ends. All bars had milled ends.

The purpose of the experiment, which was in the nature of reconnaissance only, was to study the behavior of the specimens when the reinforcing bars were definitely stressed to the yield point in order to learn whether the concrete would crack and, if so, how badly; in general, to draw conclusions as to the ability of the reinforced concrete column to take axial load with the steel under a stress at which it would definitely flow parallel to the load, or buckle if not laterally restrained. The specimens were watched particularly at three stages of the testing: (1) at the yield point of the steel; (2) when the specimen as a whole reached its yield point—that is, when it deformed rapidly without taking appreciably greater load; (3) after failure. Continuous stress-strain measurements were taken semi-autographically on all specimens. The specimens were cured under water and tested at 28 days. Results may be summarized as follows:

(1) Specimens without spiral reinforcement cracked badly when the steel reached the yield point. The 3-in.-diameter specimens, with a single ½ in. square bar at the center, failed at this stress while the protruding bars were far from being flush with the concrete; that is, the steel took very little deformation at the yield point before failure by splitting occurred. The similar 6-in. diameter specimen showed cracks at the yield point of the steel (at a total load of 12,000 lb.) but took an additional load of 46,700 lb. (58,700 total) before failure occurred. The bars were about flush at this load. The corresponding specimen, No. 6, with spiral steel, took 162,000 lb. ultimate in the case of the greased bar and 149,000 lb. in the case of the bar not greased (specimen No. 5).

(2) All specimens which had spiral reinforcement were in excellent condition when the stress in the steel reached the yield point. Under further loading, which caused the bars to gradually sink into the concrete until they were flush, some local cracks and spalling developed but in all cases the core was in very good condition.

(3) The specimens with greased bars took, with some exceptions, a slightly higher load than those with bars not greased. Specimen No. 7 with bars greased and also flush showed no cracks at failure at all and was in the best physical condition of all 6 by 12 in. specimens.

The behavior of the specimens with greased bars indicates that perfect bond between concrete and longitudinal steel, which has been a time-honored assumption for purposes of design, is not essential for developing the carrying power of columns under axial load provided some other adequate means is supplied for transferring load to the steel. In the columns of these tests the metal end bearing blocks accomplished the positive transfer of load to the projecting steel bars in the same manner as would be accomplished by properly designed butt joints or flush end plates. It should be understood that the greased bars of the tests were employed only as a device for bringing out a different aspect of fundamental column action. No implication is being made that such construction should be followed in column design.

TABLE 1.

Specimen No.	Size	Reinforcing Bars			Percentage of Spiral	Ultimate Load, lb.
		No. and Size	Percentage	Greased		
1a	6 by 12 in. cylinder	—	0	—	0	76,300
2a	"	1—½ in. sq.	0.82	No	0	58,700
3a	"	"	0.82	Yes	0	67,300
4a	"	—	0	—	3.2	169,400
5a	"	1—½ in sq.	0.82	No	3.2	149,000
6a	"	"	0.82	Yes	3.2	162,400
1	"	—	0	—	0	111,200
6	"	—	0	—	3.2	138,000
5	"	6—½ in. sq.	4.9	No	3.2	203,000
8	"	"	4.9	Yes	3.2	190,000
7	"	"	4.9	Yes	3.2	174,400 *
10	3 by 6 in. cylinder	—	0	—	0	28,600 **
11	"	—	0	—	1.4	26,800
12	"	1—½ in. sq.	3.5	No	0	10,500
13	"	"	3.5	No	1.4	38,500
14	"	"	3.5	Yes	0	11,100
15	"	"	3.5	Yes	1.4	39,200
16	"	6—¼ in. round	4.15	No	1.4	41,000
17	"	"	4.15	Yes	1.4	42,000
18	"	12—¼ in. round	8.30	No	1.4	42,500
19	"	"	8.30	Yes	1.4	61,300
20	3 by 24 in. cylinder	—	0	—	0	24,500 ***
21	"	—	0	—	1.4	25,000
22	"	1—½ in. sq.	3.5	No	0	11,000
23	"	"	—	No	1.4	32,500
24	"	"	—	Yes	0	10,100
25	"	"	—	Yes	1.4	38,300
26	"	6—¼ in. round	4.15	No	1.4	37,400
27	"	"	4.15	Yes	1.4	40,800
28	"	12—¼ in. round	8.30	No	1.4	48,600
29	"	"	8.30	Yes	1.4	50,800

*Bars flush with ends.
**Average of three specimens.
***Average of two specimens.

Whether it possesses any merit for purposes of construction is beyond the scope of this discussion.

As to the behavior of columns whose longitudinal bars become stressed to the yield point due to shrinkage and flow as well as normal design loads, it is to be concluded from these tests that columns with adequate spiral steel should be structurally safe.

The importance of proper spiral reinforcement can scarcely be over-emphasized. All reinforced column research to date seems to point to this conclusion. In this instance, too, it is evident that the spiral steel furnished the lateral support tending to keep the concrete from splitting and to restrict the splitting when it did start. By confining the concrete surrounding the longitudinal steel it enables the latter to continue to take its maximum stress even though the yield point is reached and it

allows the concrete to approach and even exceed its ultimate strength as indicated by similar specimens of plain concrete.

In conclusion it may be stated that the specimens described above were punished more severely than specimens in which the bars are flush with the surface of the concrete, due to the pressing of the steel into the concrete and the consequent greater force exerted upon the column tending to split it. It is encouraging, therefore, that the results point to the same conclusion as that indicated by previous observations and apparently forecast by the preliminary report of Committee 105, namely, that one need feel no undue concern regarding the structural safety of the reinforced concrete column as a result of the longitudinal steel being rtressed to the yield point provided the column is adequately reinforced with spiral steel. What constitutes adequate spiral reinforcement continues to be one of the major questions before Committee 105.

Readers are referred to the JOURNAL *for Dec. 1931 (Vol. 28), for further discussion which may develop. Such discussion should reach the secretary by Nov. 1, 1931.*

Discussion of Report of Committee 308

"Basis of Design for Hurricane Exposure"*

Discussion of the report of this committee, published in this Journal March 1931, Vol. 27, p. 903, was scheduled for this month. It has been deferred to September 1931, Vol. 28. —Editor

Discussion of Paper by Benjamin Wilk:

"A Comparison of Continuous with Batch Mixers in Plant Operation*"

CONVENTION DISCUSSION

H. B. Emerson: Did you have any way of measuring accurately the water used in the continuous mixer to a given amount of cement and aggregate as in the batch mixer?

Mr. Wilk: No, but we can tell from the face of the block whether or not it has the proper amount of water in it. Variations will show up noticeably—the water web mark is about as good an indication as we could have. If there is too much water the block will slump; if not enough the water web mark will not be there and the possible variation is not a great deal; John Lowell just advises me that the water variation, as so observed, is less than can be conveniently measured.

A. G. Conrow—Did you make any comparison of the number of turns in the use of the continuous mixer and in the batch mixer—the turns of the material.

Mr. Wilk—That question brings up one's ideas of a batch mixer and of a continuous mixer. The batch mixer is called upon to turn together all the cement and all the aggregate which are brought together at one loading. But in the continuous mixer I am talking about, very small portions of the aggregates and of the cement are thrown together at one time so that if for example, four cubic feet of material is divided by thirty-four of the blades, a very small proportion of material strikes a blade at a time, and it is that combination of small amounts of cement and small amounts of aggregate which, to my mind, causes the thorough-

*A. C. I. Journal, Feb. 1931; *Proceedings*, Vol. 27, p. 655. Presented by Mr. Wilk and discussed at 27th Annual Convention, Milwaukee, Feb. 24-26, 1931.

ness of the mixing and which it looks to me is very difficult to get in a batch mixer. If you are going to put in eight cubic feet of aggregate and on top of that throw one cubic foot of cement, it is going to take quite a while to mix it as thoroughly as in a continuous mixer where small portions of the material are thrown together at one time in the mixer trough.

Discussion of Paper by H. F. Faulkner and R. R. Hubbard:

"Design and Control of Concrete for Diablo Dam*

DISCUSSION BY F. R. MCMILLAN

It is very gratifying to have in the *Proceedings* of the American Concrete Institute this description of the concrete and of the unique methods of placing used in the Diablo dam. In both of these features this project provides significant departures from the practice which has become almost standard for large dam construction. The speaker had the good fortune to visit this structure in company with the authors and Mr. Carver, and can testify to the success which attended these methods.

The use of large sized aggregate for structures of this size has

*A. C. I. Journal, February 1931; *Proceedings*, Vol. 27, p. 529. In the absence of the author, the paper was presented at the 27th annual convention, Milwaukee, Feb. 24-26, 1931, by F. R. McMillan, who called attention to the outstanding features of the work and presented discussion as here published.

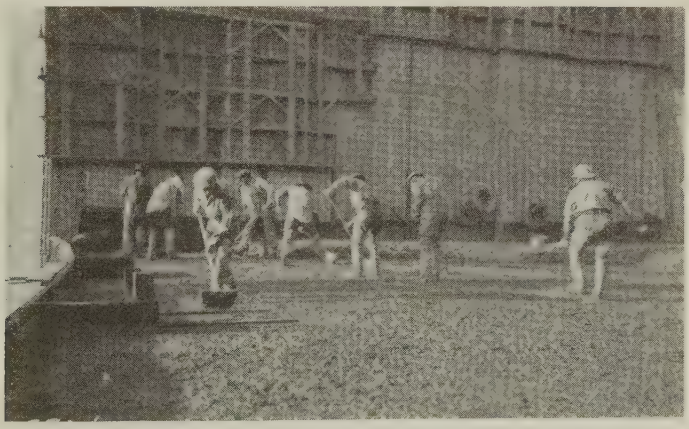


Fig. 1—Brooming surface with mortar just before placing a layer of concrete



FIG. 2—COMPLETED DAM, DIABLO CANYON

become so nearly universal in recent years that the limitation of this aggregate to a maximum size of 4 in. will, no doubt, attract considerable attention. The authors have set forth quite clearly the reasons for this limitation. While advocates of the use of "cobbles" up to 8 or 10 in. can show successful results to offset some of the claims, instances can be cited of results not as satisfactory as at Diablo Dam. The mixtures with 4-in. maximum size showed up particularly well under the method of placing used which, as pointed out before, was also an innovation in constructions of this kind.

Figs. 4 and 5 of the paper show the belt conveyors suspended from the tower and boom. Both towers in Fig. 4 are seen to be rigged in this manner. Fig. 6 shows the "elephant trunk" through which the concrete was deposited. Something of the character of the mixture can be seen from this view. The absence of a great range in sizes, together with the method of placing, practically eliminated all undesirable segregation and required a minimum of handling and puddling.

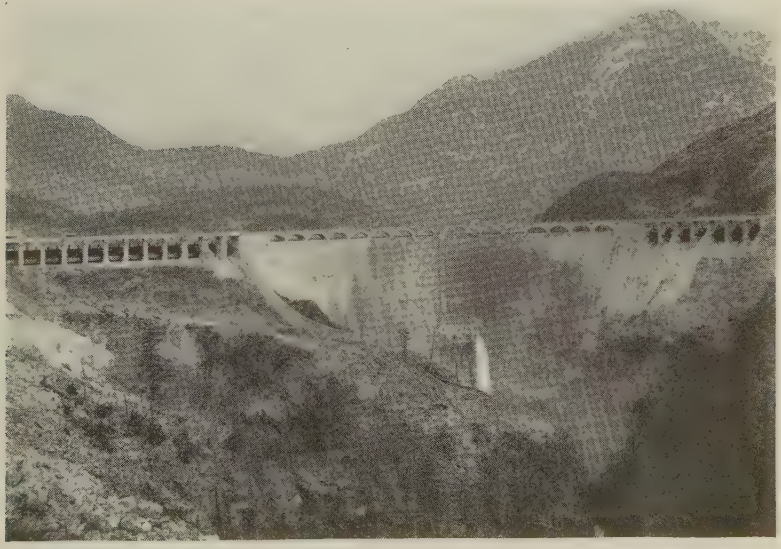


FIG. 3—COMPLETED DAM. DARKENED PORTION OF FACE IS WET WITH SPRAY FROM STREAMS PASSING THROUGH NEEDLE VALVES

Three views are here added to those presented by the authors with the paper.

Fig. 1 shows the brooming of the surface with mortar just before placing a layer of concrete, as described by the authors. Placing the concrete in thin layers, which can be seen in this picture, insured a thorough puddling and at the same time reduced tendency to the formation of laitance.

The success of the method, of course, can be judged only by the final product. Figs. 2 and 3 show the completed dam with the reservoir practically full. As brought out in the paper there is some leakage on some of the horizontal joints. This leakage however, is restricted to a very small number (less than 1 in 20, it appears from the pictures) and to very small amounts.

This tightness is extremely favorable in comparison with some of the better constructed dams of comparable dimensions in which 8 and 10-in. aggregate was used. The fact that so large a proportion of the joints is perfectly tight is evidence that the mixtures and methods were entirely adequate. Such defects as are noted only emphasize the need for constant attention to all the details of preparation of surfaces and placing the concrete to obtain perfect results.

Discussion of Paper by J. L. Mensch:

"Composite Columns"*

BY FRITZ EMPERGER†

In the infancy of the reinforced concrete era two different view-points opposed each other. One of them leaned towards steel construction and considered concrete as a filler between rigid reinforcements. The other would improve the concrete by flexible reinforcement. Round steel bars finally won general recognition as reinforcement as the question of increased tensile strength became more important. It was soon clear that the round steel bar did not represent the most suitable form of reinforcement for concrete in compression, and a number of limitations have therefore been specified.

These conditions were present during the time I spent in North America. In the beginning of the reinforced concrete era (1890-1896) a city engineer of Philadelphia, simultaneously with me, built a structure with girders of 20-ft. free span, and Ransome in San Francisco built girders of equal span. At that time nobody had the courage to use larger spans with ordinary reinforcing bars. In 1893 to 1894 I designed and built an arch bridge of 70-ft. span in Cincinnati, one of 100-ft. span in Stockbridge, Mass., and also used the Melan system in a number of other structures, which became the few representatives left for the rigid reinforcement which today is seldom used.

After a stay in Paris in 1900, I returned to Vienna and in 1902 I embedded a number of steel columns in concrete. The test results are published in my paper on column construction in *Beton und Eisen*, 1907, page 172, and in *Forscherarbeiten*, Heft 8, Berlin, 1908.

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*A. C. I. Journal, Nov. 1930; *Proceedings*, Vol. 27, p. 263.
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The difference between round steel bars and structural steel shapes as reinforcement for columns should not be considered merely as a difference in form; here occurs a fundamental distinction as regards difference in determination of the permissible load. The permissible load for columns of ordinary reinforced concrete is practically always determined from the equation:

$$P = (A'_c + A_s n) f_c \quad \ldots\ldots (1)$$

Where A'_c = the area of the concrete in the column
A_s = effective cross section of steel.
f_c = permissible compressive stress in concrete.
n = 15.

We see, therefore, that the permissible load is determined by the specified compressive stress in the concrete f_c. This stress appears to be so important that the whole construction depends upon it. It requires no great intelligence to show that this view-point has little justification, since the permissible compressive stress in the concrete can be exceeded without any dangerous results, and safety against failure is the only important item.

The other view-point, however, uses the ultimate strength as the determining factor, and demands such an arrangement of both materials that they work together and that both will be completely utilized until failure. The load at failure of the column is then:

$$P_{failure} = A'_c f'_c + A_s f'_s = P'_c + P'_s \quad \ldots\ldots (2)$$

Where f'_c = ultimate compressive strength of concrete
Where f'_s = ultimate compressive strength of reinforcement.

By means of a given factor of safety we arrive at the permissible load. This load is arrived at still more simply from the equation,

$$\text{Permissible load } P = A'_c f_c + A_s f_s = P_c + P_s \quad \ldots\ldots (3)$$

When both materials have their own factors of safety, the column has an average factor of safety which naturally depends upon which material predominates and which approaches the factor of safety for that material. In the beginning of the reinforced concrete era the possibility of full cooperation was questioned, and I made the tests of steel columns shown in Figs. 1 and 2 with and without concrete in order to show plainly how strengths are added. These figures are found in the fourth edition of my

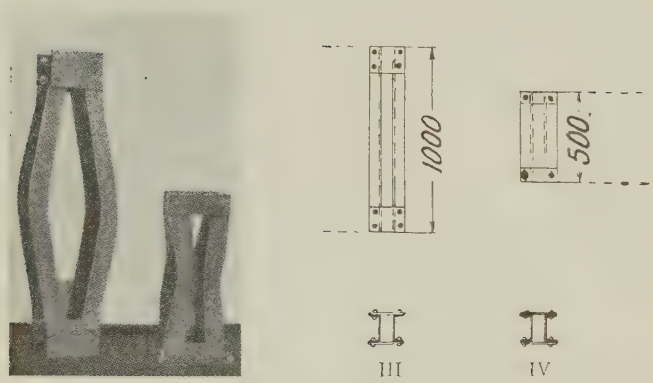


FIG. 1

handbook for reinforced concrete. It has been maintained that the materials will work together up to only 2 per cent reinforcement, but I have made tests with 12 per cent and believe that 20 per cent and more would work just as well with sufficient lateral reinforcement and sufficient strength of the concrete.

In 1910-1911 I made further tests with spirally reinforced steel and cast-iron columns and the results are published in Heft 3 of Reports of the Austrian Committee for Reinforced Concrete (see pages 57 and 116 *Beton und Eisen*, 1912), and found that by means of the lateral reinforcement even the highest compressive strength of steel was fully utilized in adding strength. From the deformation I derived the relation between the elastic carrying capacity and the strength addition, and, as at that time I could find no cheaper iron material than cast iron, I took especial interest in this material. Even though the war had already lasted for one year I was able to present the results of our work at the International Engineering Congress in San Francisco, 1915, and showed that the relation presented in Fig. 3 of the deformation of a column may be considered as the sum of two deformation curves. When the two areas in a reinforced concrete column correspond to A_c and A_s, and the corresponding load curves are P_c for the concrete and P_s for the steel, the addition of the two curves gives the actual deformation curve of the reinforced concrete column in the curve $P_c + P_s$.

This is a long-drawn-out introduction of the question which received almost 12 years interruption by the war and the hard times following the war, but it was necessary in order to go into the investigation which has been reported by Mr. Mensch.

In the first place I may say that it has not surprised me that the American cast iron shows so much lower strength than that used

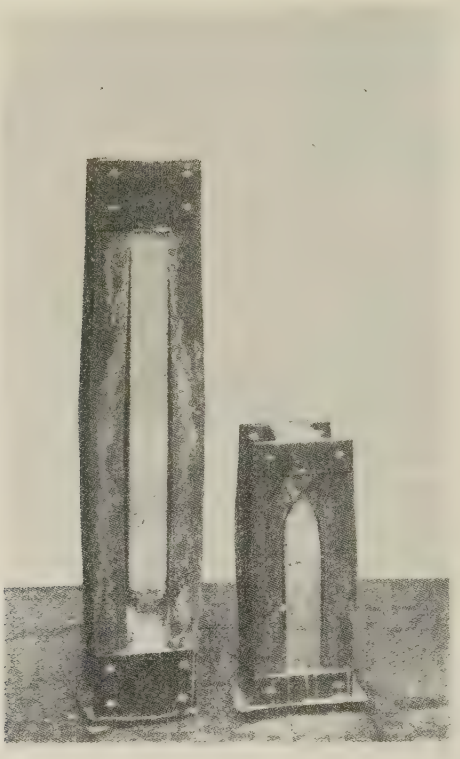


FIG. 2

in my Austrian tests. I have had the same difficulty with the German cast iron. The German and also the American foundries have no interest in producing a cast iron of high compressive strength. This is only a question of composition of the iron, primarily the carbon content. It is important to point out this fact since the question of the cost of producing cast iron with a compressive strength of 150,000 lb. per sq. in. plays no role even

though the ordinary cast iron has a strength of only 75,000 lb. per sq. in. In the specification (see Tetmayer) the minimum strength required for steel for buildings is always 100,000 lb. per sq. in. The Austrian foundries deliver cast iron which without special requirement has 150,000 lb. per sq. in. But in Germany one may get by with a material having a compressive strength approximately that of ordinary steel, that is less than 75,000 lb. per sq. in. (See the deformation curves published by Bach in Heft 3 of the Reports of the Austrian Committee).

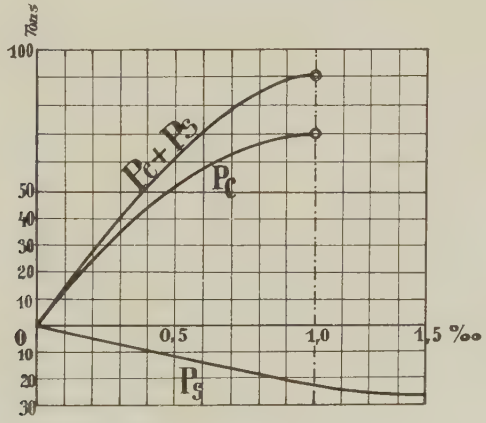


Fig. 3

Nevertheless it is important to note that the foundries have no interest in producing for the ordinary machine casting a material of high compressive strength, which naturally is much harder to work. When we in Austria have a cast iron of such a high compressive strength, it is due to the charge present in the foundries. When one in Germany wants a cast iron of high compressive strength he will have to find a foundry which happens to have a suitable composition. But if he requires a certain compressive strength from the foundry the operators do not know what price to charge for it. Since the use of cast iron for building purposes has practically stopped, they will have to make a special

charge for the work. For industrial use I believe that the situation for cast iron is the same in North America, and a study of the deformation curve would show that. The deformation curve for this kind of cast iron is quite flat and a high ultimate strain is required for the concrete, that is, a large amount of spiral reinforcement of high strength steel, in order to utilize the relatively low compressive strength.

In order to understand what important economical advantage the use of cast iron had in Austria before the war it must be remembered that I easily managed to utilize 100,000 lb. per sq. in. of a cast iron having a strength of 150,000 lb. per sq. in. by using a proper amount of lateral reinforcement, and could make use of permissible stresses up to 20,000 lb. per sq. in. However, when I used ordinary grade steel in columns I was governed by the specification which permits a maximum stress of 570 lb. per sq. in. in the concrete and 8,500 lb. per sq. in. in the steel, so that the utilization of the cast iron to double this amount, and more, would have reduced the reinforcement to one-half and less.

I want to mention that the conditions after the war have caused a considerable change. In Europe the cost of cast iron has increased relatively more than the cost of steel, and the most important factor is that while prior to the war we had to use a steel which at most had a yield-point stress of 34,000 lb. per sq. in., and a corresponding working stress of 14,000 lb. per sq. in., we have at present steel grades on the market with a yield-point stress of 47,000 to 53,000 lb. per sq. in. and permissible stresses up to 22,000 lb. per sq. in. Interest in the use of cast iron has therefore decreased considerably in Europe, especially on account of the official codes. The officials are not inclined to permit anything new, and certainly not anything they believe questionable. Cast iron can without doubt be poorly made and good cast iron requires careful manufacturing and rigid control.

The new methods of making cast iron produce excellent material, but the knowledge has not yet been acquired by the proper officials. It is illuminating in this respect that I have designed and built a number of important buildings and even

official projects in Austria and Czechoslovakia with columns of spirally reinforced concrete and cast iron. In Germany, however, where I designed arch bridges of this material with spans up to 250 feet, I was not permitted to make a single column of this kind, the only reason being the stubbornness of the building officials.

I cannot, however, explain the error limiting only to cast iron the viewpoint presented in the addition method given in equations 2 and 3. I have always made designs both with cast iron and with steel and left it to the contractor to decide which method to choose. We have even made far-reaching tests with high strength steel for the purpose of obtaining specifications for columns of steel skeletons. This work was published in March, 1931, as Heft 12 of the Reports of the Austrian Committee for Reinforced Concrete. In the following a short summary of these tests is given:

(1) The average deformation of ordinary concrete in a column amounts to 0.1 percent. This value will not be exceeded in a column with ordinary ties, showing that at failure the longitudinal reinforcement can not be utilized to more than 29,000 lb. per sq. in. whatever the yield point stress of the steel.

(2) For spirally reinforced concrete columns the shell outside of the spiral deforms with the column up to a deformation of 0.2 per cent. If only a small amount of spiral reinforcement is used (as is the case when the spiral is not intended to increase the strength but only for the utilization of the yield-point stress of the steel) the total cross section of the column can be used in the computation. It is not necessary to use only the area of the concrete within the spiral as has been common practice up to the present time.

(3) By the use of rigid structural shapes for reinforcement or of independent cast-iron columns, a direct increase in strength is obtained and the use of a small amount of spiral reinforcement will raise the strength as for the round iron reinforcement.

(4) These tests have recently proved the correctness of equations (2) and (3) regarding the "law of addition." With our limited means we managed to make 42 column tests during the summer of 1930. We aimed at a few typical examples and showed

how the strength of the steel column can be increased by 72, 108 and 172 per cent, by a simple embedment in concrete spirally reinforced. For such a skeleton half of the reinforcing steel can be saved by this arrangement which is so necessary with respect to safety against damage by fire. I have to my surprise read a circular from the American Institute of Steel Construction[1] wherein complaints are made against the fire insurance companies in the United States for specifying a thick concrete shell for steel frame buildings. This shell is considered only as dead weight. If this is dead weight they themselves are to blame because they evaluate the concrete so low. If they recognize that the concrete shell contributes to the strength, they can save half the steel weight by the utilization of an arrangement which is required anyway and so actually costs them nothing. I believe that this method will lead to a compromise between the competing methods of steel and reinforced concrete constructions. This method may be carried back to the early days of my stay in the United States nearly 40 years ago and I am glad to have been able to contribute my small share to the development of American building construction.

BY R. SALIGER*

Having made during the last years a great number of tests the Laboratory of the Vienna Technical College is in a position to add materially to the knowledge on composite columns. All tests were made very carefully as regards materials and methods in order to obtain that high degree of uniformity necessary for scientific comparison. The tests were made with columns of Dr. Bauer's system with cast iron skeleton and with hard steel bar skeleton.

(*A*) *Cast iron skeleton*—During 1928 tests were made on 48 columns partly of 47 in. and partly of 109 in. length. The columns were square or round, the hooping following the square or cylindrical form on the outside. The cast iron skeleton consisted of 4 bars having a total area of 5 sq. in., 10 sq. in. and 15 sq. in. respectively. The bars were of rectangular section and were held together by cast iron rings thus forming one rigid skeleton. The

[1] "Fireproofing Structural Steel," by Jack Singleton, published by American Institute of Steel Construction, 1929.

*Prof. Dr. ing., Vienna University Technical College.

cast iron core was from 4.6 to 15.3 per cent of the total area inside the hooping. The spiral consisted of round mild steel wire of 0.2 in., 0.28 in. and 0.4 in. diameter respectively at 1 in. to 4 in. pitch. The spiral was from 0.5 to 4 per cent of the hooped area. The concrete mixture was 1 part cement to 4 parts sand, cube strength being 2,870 lb. per sq. in. and cylinder strength 2,020 lb. per sq. in., age 15 days.

The ultimate strength of the cast iron core was 113,600 lb. per sq. in. Tests of all spiraled columns showed that with very slight variations the ultimate strength is given by the formula:

$N_{ultimate} = F_k \sigma_p + F_g \sigma_g + 2.2 F_s \sigma_s$

F_k = hooped concrete area

F_g = area of case iron core

F_s = equivalent area of spiral reinforcement.

σ_p = cylinder strength of concrete 2,020 lb. per. sq. in.

σ_g = utilizable resistance against compression of cast iron 68,160 lb. per sq. in.

σ_s = yield point of spiral wire.

According to results obtained the total resistance to compression of the cast iron core was utilized up to 63 per cent. The long columns carried at failure 15 per cent less than the short ones.

(*B*) *Hard steel bar skeleton*—During 1929 and up to 1931 four series of tests of columns from 47 in. to 120 in. long were made at the Vienna and Berlin laboratories. (Technische Versuchsanstalt Wien und Material Prüfungsanstalt, Berlin).

The spiral hooping of all columns was 12 in. in diameter, the outside diameter of the columns being 13.4 in to 13.8 in. The spiral consisted of round wire from 0.2 in. to 0.28 in. diameter at 1 in. to 2 in. pitch. Flat iron hooping was also used. The spiral was from 0.5 to 2.0 per cent of the hooped area. The hard steel bars had a tensile strength from 113,600 lb. to 142,000 lb. per sq. in and a yield point in compression of 106,500 lb. per sq. in. The diameter of the longitudinal bars was from 1.2 in. to 1.7 in.; their number varying from 4 to 6 in each column. The metal area of the core represented from 3.8 to 11.2 per cent of spiral area.

There were 10 to 20 columns tested in each series. By selecting sand and gravel of good quality and by careful mixing great uniformity and quality of the concrete was obtained throughout. Otherwise the results would scarcely have formed a sufficiently reliable base for comparison.

We may arrive at the ultimate strength of the columns, with very slight variations, as follows:

$N_{ultimate} = F_k \sigma_p + F_e \sigma_e + 2.9 F_s \sigma_s$

σ_e = yield point of the longitudinal steel bars embedded in hooped concrete = 113,600 lb. per sq. in.

The efficiency of hooping turned out to be 2.9 times the efficiency of longitudinal reinforcement with material of the same quality and weight. The utilizable resistance against compression of hard steel core was 113,600 lb. per sq. in., thus being greater by 6 per cent than the actual yield point in compression of the naked steel bars.

By comparing the results of four series of tests the following conclusions were arrived at:

1. The strength of the metal core will be fully utilized if the spiral represents 0.5 per cent of the hooped area.
2. Increasing the hooping results in an increase of the resistance of hooped concrete only.
3. Columns 120 in. long show a carrying capacity 15 per cent smaller than that of columns 47 in. long.
4. The utilizable resistance to compression of columns with steel bars welded of two pieces is 10 per cent less than that of columns with bars in one length, not welded.
5. The carrying capacity of the columns is determined by the combined resistances to compression of the concrete core, of the metal core and the tensile strength of the spiral.
6. With small loads the hooped concrete and the metal core remain one solid mass. With big loads the outer layers crack and fall off sooner or later before the stress at failure is reached, according to strength of the metal core.

(*C*) *Structural steel skeletons*—Recently 10 columns with structural steel skeletons, spirals and concrete were tested. All columns were 120 in. long, had octagonal form and 13.8-in. out-

side diameter. The structural steel core consisted of two channel sections, 4 in., 6 in. and 7 in. in depth, held together by gusset plates. The spirals consisted of bars 0.2 in. and 0.28 in. thick at 2 in. pitch. The yield point of structural steel was from 35,220 to 37,200 lb. per sq. in. The tensile stress of the spiral wire was from 35,500 to 39,800 lb. per sq. in.

The concrete mixture of 1:5 resulted in a cube strength of 2700 lb. per sq. in. and a cylinder strength of 2100 lb. per sq. in. at 30 days.

All tests gave extremely uniform results regarding load at cracking of outer layers and load at failure. Loads at cracking were from 4.7 to 13.1 per cent under maximum loads.

At failure the stress in the concrete was estimated equal to the cylinder strength. Stresses in the structural steel were 10 per cent greater than the above mentioned yield point. No sideways buckling was observed.

Most columns are so short that the spirally reinforced concrete would fail before the column buckled. The strength of such a column would be the summation of the strengths of the spirally reinforced concrete plus the load borne by the cast iron at the ultimate strain of the spirally reinforced concrete, unless the per cent of cast iron is unusually high.

BY JOHN TUCKER, JR.*

The concrete column reinforced with spiral and cast iron that may be justly termed the Emperger column has proven of merit in actual construction. Mr. Mensch is to be congratulated upon his sustained interest in presenting information and obtaining data on this type of column.

The tests that have been made upon columns of this type unfortunately have given insufficient data upon which to obtain constants based on a sound theoretical analysis. In particular, the control specimens that would give the strength of the concrete in the reinforced columns are very meager. The ultimate strain and the modulus of elasticity of the spirally reinforced concrete up to the ultimate strain should be known. The Emperger columns tested in the United States have had flat ends bearing directly upon plates on the platens of the testing machine. The amount of

*Bureau of Standards ,Washington, D. C.

motion of the spherical bearing under such conditions is small, and the tests approximated the condition of fixed ends. The movement that did occur[1], however, confused to a considerable extent the conclusions that might be drawn from the tests.

If, instead of empirical equations that hold for each set of tests only, as have been obtained, completed data were secured upon columns tested with definite end conditions, the critical length (for buckling) could be computed by the Engesser-Considere theory[2]. It is regretted that this splendid column theory which has been experimentally confirmed[3] for cast iron has not been applied to the interesting Emperger type; and it is also to be regretted that the theory is not familiar to more engineers in the United States.

Mr. Mensch gives values of .001 and .01 for the ultimate strain of plain and spiral reinforced concrete; these values are approximately correct but may be sufficiently misleading to warrant correction.

The average ultimate strain of unreinforced concrete[4] is approximately .0014 in. per in., the individual readings varying from .0010 to .0018, approximately. The ultimate strain of the spiral reinforced column is under .008 in. per in., some experimentally obtained values being as low as .003. The test data indicate that the higher the per cent of spiral reinforcement the greater the ultimate strain.

AUTHOR'S CLOSURE

Prof. Dr. R. Saliger contributes the result of about 100 tests on composite columns. It is gratifying to note that not once in his discussion does he make use of the ratio of the moduli of elasticity of metal and concrete, commonly denoted by n. The reason is that the strength of reinforced concrete cannot be explained by the use of n which changes in value at every change of loading.

This discussion contains also a clear cut statement that the action of the spiral hooping is proportional to the yield point of the wire, a truth appreciated by very few engineers. The tests on

[1]See for example Bureau of Standards Technologic Paper No. 122.
[2]See Meyer, Die Knickfestigheit, 1921, p. 69.
[3]See for example Die Knicksicherheit an beiden Enden gelenkig gelagerter Stabe aus Gusseisen. T. Yuki—Sendai Higher Technical School, Japan, June 11, 1930.
[4]Refer to Talbot, Bul. No. 10, University of Ill. Exp. Sta. Withey, Bul. No. 666, University of Wisconsin Eng. Series, Bureau of Standards Technologic Paper 122 and many other papers.

hooped columns with square spirals which proved to be nearly as efficient as round spirals are new to American practice.

Although the cast iron used in Dr. Saliger's tests was of lower strength than that used by Dr. Emperger the ultimate stress per sq. in. was higher than in the American tests because of the very low value of the l/r of the cast iron skeletons. The series B of Dr. Saliger's tests with skeletons of high carbon transmission shaftings is very instructive in view of the fact that many city codes and specifications do not permit higher stresses on high carbon bars than on medium steel bars. After giving the hooped concrete in this series all possible credit for the strength it contributed, the remaining part of the ultimate load divided by the area of the steel skeleton amounted to 113,600 lb. per sq. in., which is 6 per cent larger than the yield point. Here we attribute to steel bars a stress of only 30,000 lb. per sq. in. at the ultimate load and consider high carbon bars of no advantage in columns. These tests ought to be effective in dispelling this error.

Series C with structural steel cores gave nearly identical results with the columns 11 to 14, made by the writer, and shown in Table 4.

Mr. Molitor's discussion[1] is answered by Dr. Saliger's and Dr. Emperger's discussions.

John Tucker, Jr., who did the actual testing of the Emperger columns at the Bureau of Standards' Pittsburgh Laboratory in 1916, voices regret in his discussion that no attenpt has been made to obtain data for a sound theoretical analysis of composite columns. The writer wishes to emphasize that, notwithstanding the fact that thousands of extensometer tests have been made on simple concrete beams, (which tests show to be nearly twice as strong as the accepted theory teaches), we have no reliable data to explain their behavior, and all the extensometer tests made on spiral columns have not taught American engineers to use only high elastic limit steel for spiral and vertical reinforce-

[1] A. C. I. Journal, March 1931; *Proceedings*, Vol. 27, p. 947.

ment, and that all our positive and valuable knowledge of the strength of materials consists of more or less empirical data, derived from practice or tests. The modulus of elasticity of concrete varies greatly after ½ of the ultimate strength is reached, as it does for cast iron, while the common theory which assumes a constant modulus of elasticity leads engineers astray and prevents them from adopting new combinations of materials.

Discussion of paper by M. O. Withey:

"Some Long Time Tests of Concrete"*

AUTHOR'S CORRECTION

P. 549, in the ninth line from the bottom of the page, change *25 deg.* to *— 25 deg.*

P. 559, in the first column of Table 5, third line below headings, change *expansion* to *contraction.*

BY W. K. HATT

Professor Withey's paper on "Long Time Tests of Concrete" contains Fig. 8, showing the relation between volume-change and loss or gain of water in curing.

The diagram in my Fig. 1 shows these relations from the several series of tests performed in the Laboratory for Testing Materials of Purdue University during the last two or three years. The notations on the figure give information of the nature of the aggregate, the concrete mix, etc.

It is apparent that the law connecting change of weight, resulting from gain or loss of water, and change of length, obtained by strain gage measurements, is not a straight line relation. This relation varies with the richness of the mix.

The duration of constant immersion or constant air drying as represented in Fig. 1 varied from 24 hr. to 130 da. at which latter time the specimens had closely approached a period of equilibrium.

The zero hour of beginning the measurements was 24 hr. subsequent to a period of 24-hr. curing of specimens in a damp atmosphere.

*Presented at the 27th Annual Convention, A. C. I., Milwaukee, Wis., Feb. 24-26, 1931. A. C. I. Journal, February 1931; *Proceedings*, Vol. 27, p. 547.

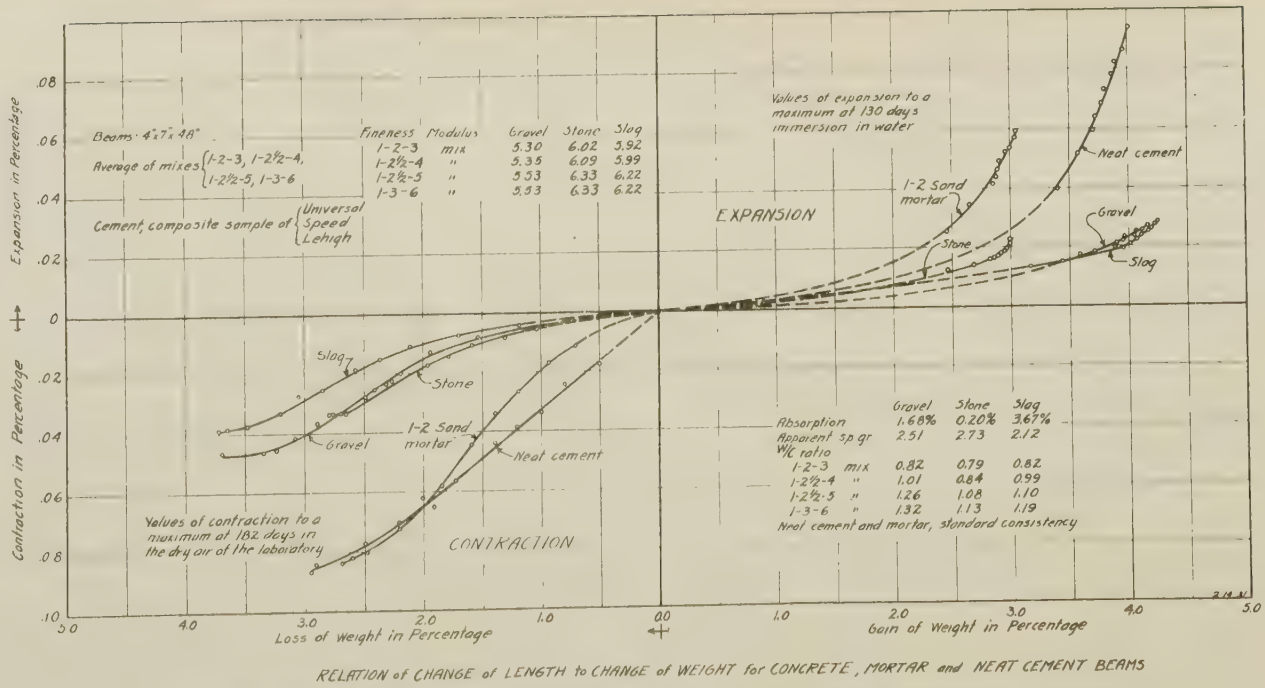


FIG. 1 RELATION OF CHANGE OF LENGTH TO CHANGE OF WEIGHT FOR CONCRETE, MORTAR AND NEAT CEMENT BEAMS

CONVENTION DISCUSSION

M. O. WITHEY—I would like to suggest that the coordinates to this curve, Fig. 1, are not the same as shown in my Fig. 8. In my Fig. 8, I show loss of water in gallons per sack of cement. Professor Hatt's figure shows loss or gain as a percentage of the weight of the sample of concrete. If his figure is changed so that the coordinates agree with those shown in my Fig. 8, I think there will be more marked agreement in the two sets of data.

BY DUFF A. ABRAMS

A study of concrete strengths by the same organization and on the same cement over a period of 20 years is unique in engineering literature. I wish to congratulate Prof. Withey on this excellent report. I hope he will be here to present the final report when the 100-year tests are completed.

At the Lone Star Research Laboratory in New York City, we have carried out during the past few months, an extensive series of compression tests of concrete using:

2 cements
6 mixtures
3 consistencies for each of above conditions
6 ages at test (others to follow)

Our tests have a direct bearing on certain phases of Prof. Withey's work. I wish to speak particularly of the age-strength relation for concrete made of cements of different characteristics using a wide range of mixes and consistencies and to call attention to other significant phases of these tests.

The speaker pointed out in 1918[1] that the strength of concrete was a function of the logarithm of the age at test. This conclusion was based on tests from many American and European laboratories which were available in the literature at that time. The recent tests referred to above have shown that this logarithmic relation is not general for all cements, mixes and consistencies. The logarithmic relation was true for the cements and mixtures used a dozen years ago, but does not apply to modern cements

[1] "Effect of Age on Strength of Concrete" by Duff A. Abrams, *Proc.* A. S. T. M. Part II, 1911, p. 317.

and the richer or drier mixtures (that is concrete of the lower water-ratios).

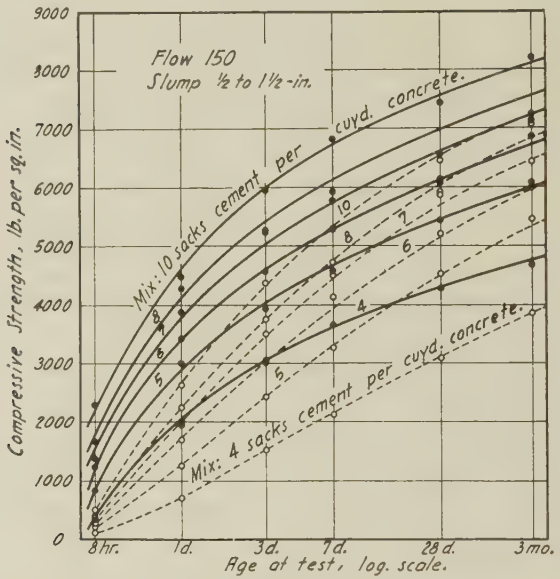


FIG. 1—AGE-STRENGTH CURVES FOR CONCRETE OF FLOW 150 PER CENT

Compression tests of 6 x 12-in. cylinders cured in moist room.
Sand and gravel aggregates.
Each point is the average of 2 tests.
Solid curves, High-Early-Strength Portland Cement.
Dotted curves, Ordinary Portland Cement.

Figs. 1, 2 and 3 show age-strength curves for concrete ranging in cement content from 4 to 10 sacks per cu. yd. of concrete. Compression tests were made on 6 x 12-in. cylinders cured in moist room. Each figure is for concrete of a definite flow as measured by 15 ½-in. drops of the 30-in. flow table. The solid lines represent the high-early-strength cement; the dotted lines, ordinary portland.

Many will recall that formerly little attention was given to concrete strength at ages earlier than 28 days; later we became more venturesome and made tests at 7 days; lately the test

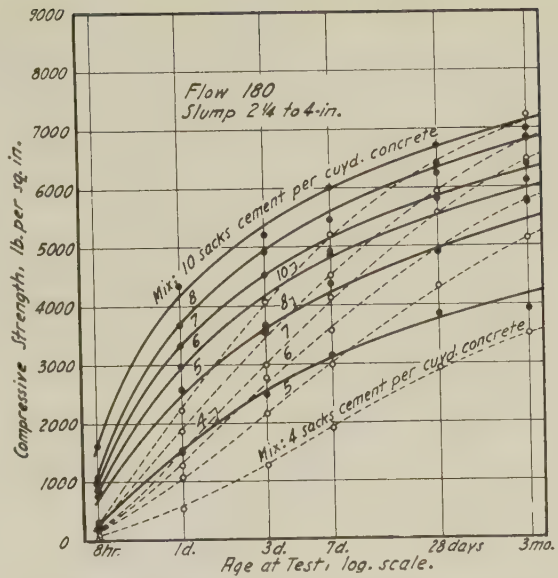


FIG. 2—AGE-STRENGTH CURVES FOR CONCRETE OF FLOW 180 PER CENT

Compression test of 6 x 12-in. cylinders cured in moist room.
Sand and gravel aggregates.
Each point is the average of 2 tests.
Solid curves, High-Early-Strength Portland Cement.
Dotted curves, Ordinary Portland Cement.

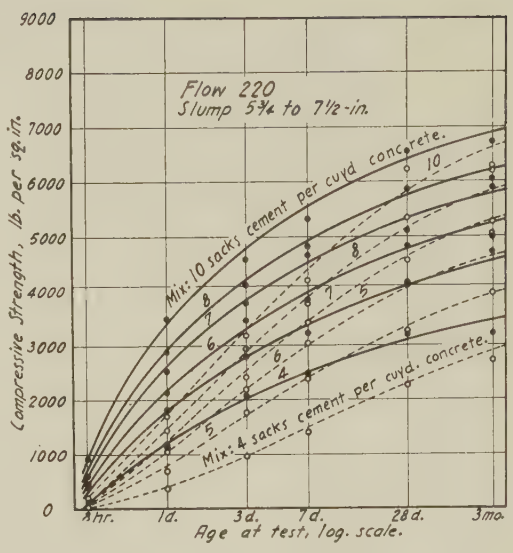


FIG. 3—AGE-STRENGTH CURVES FOR CONCRETE OF FLOW 220 PER CENT

Compression tests of 6 x 12-in. cylinders cured in moist room.
Sand and gravel aggregates.
Each point is the average of 2 tests.
Solid curves, High-Early-Strength Portland Cement.
Dotted curves, Ordinary Portland Cement.

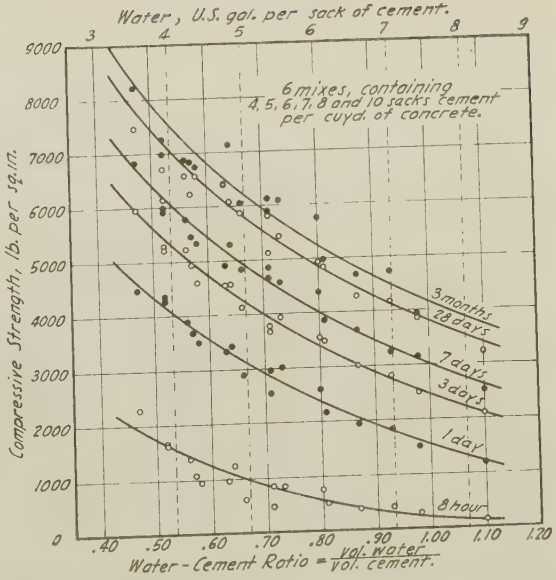


FIG. 4—WATER-RATIO—STRENGTH CURVES FOR HIGH-EARLY-STRENGTH CEMENT

Compression tests of 6 x 12-in. cylinders cured in moist room.
Each value is average of 2 tests.
Same tests as in Figs. 1, 2 and 3.

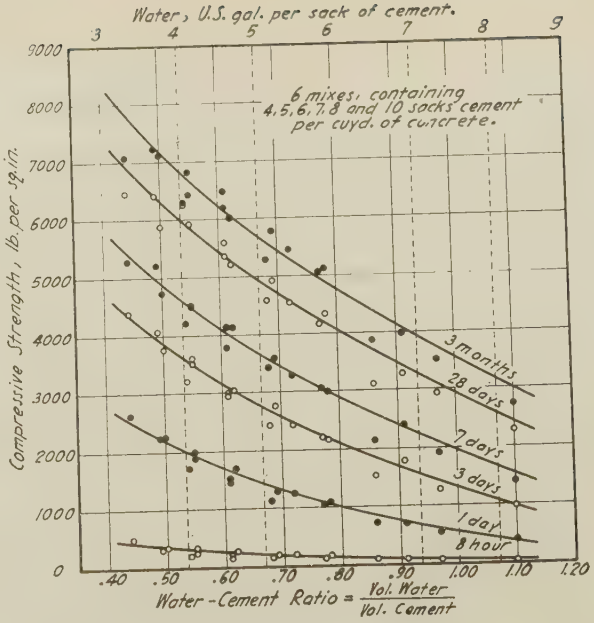


FIG. 5—WATER-RATIO—STRENGTH CURVES FOR ORDINARY PORTLAND CEMENT

Compression tests of 6 x 12-in. cylinders cured in moist room.
Each value is average of 2 tests.
Same tests as in Figs. 1, 2 and 3.

periods have been reduced to 3, 2 and 1 day and now tests are regularly made as early as 8 hours.

It is only the leaner mixes of ordinary portland cement which give essentially straight lines when plotted to a logarithmic scale of ages. It is interesting to note that relation holds for test periods as early as 8 hours. As soon as we get into the richer mixtures and drier consistencies (low flow) the lines curve upward sharply and depart materially from straight lines. The modern cements and richer mixtures have a rate-of-increase-in-strength at early periods which is much higher than the older cements and leaner mixtures which were prevalent a dozen years ago. This high rate has the effect of throwing the curves much higher on the scale—too high for the logarithmic relation to hold.

Figs. 4 and 5 are the same tests plotted with reference to the water-ratio of the concrete; separate curves are drawn for each age. These curves are in the form with which you are familiar and show that both cements give water-ratio strength curves of exactly the same form. The only difference is in the age at which a given cement reaches a designated position on the diagram.

If now we plot the curves in Figs. 4 and 5 to a logarithmic scale of strengths we find a very interesting relationship as shown in Fig. 6. All of the water-ratio curves become straight lines regardless of type of cement or age at test and they all intersect the ordinate for zero water at a single point marked A.

The equation of all these curves may be written in the form

$$S = \frac{A}{B^x}$$

where S = the compressive strength
x = water ratio (an exponent)
A and B are constants

This is the same equation which I used in 1918 in the original publication of the water-ratio law[2] for concrete strength. It was then stated that "A and B are constants which depended on the quality of the cement used, age of the concrete, curing conditions, etc." It was my original thought that each cement had a dis-

[2] "Design of Concrete Mixtures" by Duff A. Abrams, Bulletin 1, Structural Materials Research Laboratory, Lewis Institute, Chicago.

tinctive value for A, however these tests give an entirely different view and suggest that A is an *invariant* which is common to all portland cements and that B is quantity which varies with quality of cement and the age at test. As the quality of the

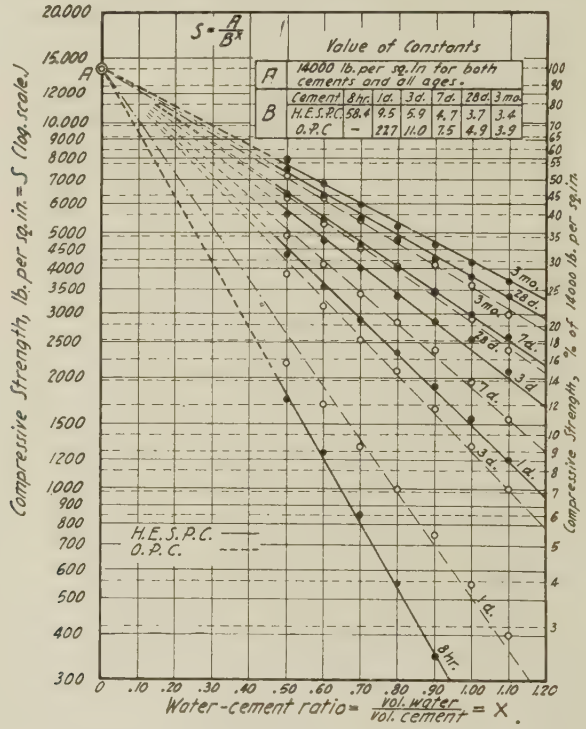


FIG. 6—WATER-RATIO—STRENGTH CURVES FOR CONCRETE
Same curves as in Figs. 4 and 5, strengths plotted to a logarithmic scale.

cement or age at test increases the value of B becomes smaller. B is the slope of the curves on this diagram.

All of the water-ratio strength curves in Fig. 6 are seen to be rotating about the point A. The quality of cement (as measured by B) simply *dictates the age at which a given strength will be reached.* In other words cement quality may be defined in terms of a rate of increase in strength at early ages.

That A is a most interesting invariant is indicated by the fact that it is the same (14,000 lb. per sq. in.) for the modern cements

as for the cements tested 15 years ago. It is no accident that Prof. Withey's tests give this value for A when plotted on our Fig. 6.

The point A is in itself an absurdity, since it is a concrete strength for a zero water-ratio; however it does have a real significance in indication of a "ceiling" or ultimate strength which all concretes, of whatever grade of portland cement, mixture or water-ratio approach, so long as they do not dry out.

My interpretation of the significance of A is that it is impossible to mold concrete under gravitational forces only which will develop strength greater than 14,000 lb. per sq. in. This, of course, applies to 6 x 12-in. cylinders and to the particular routine of testing which is now commonly followed. The consolidation of concrete under pressure appears to give a much higher "ceiling."

The tests in Bulletin 1 referred to above gave $B = 7$ at 28 days for ordinary portland cement. The table in Fig. 6 gives for ordinary portland cement $B = 4.9$ at 28 days and $B = 7.5$ at 7 days. The modern "ordinary cements" give strengths at about 9 days equivalent to the 28-day strength of the cements manufactured 15 years ago; the high-early-strength cement gives the same strength at about 2 days.

Each point on Fig. 6 represents some combination of cement quality, water-ratio and age. These are the 3 factors which go to make up concrete strength.

The scale at the right of Fig. 6 enables us to read off the percentages of the "ceiling" strength reached at any water-ratio or age; for example with a water-ratio of 0.80, ordinary portland cement has 7 per cent of its strength at 1 day; high-early-strength cement has 17 per cent. At 7 days we have 21 per cent and 28 per cent. For higher water ratios the spread between the two cements is greater and at lower water-ratios the spread is less than the values given.

The *time element* is of fundamental importance and appears to be the only real difference in concretes made and tested under standard conditions and cured damp.

The exact causes for this definite "ceiling" in concrete strength of portland cements are not clear. There are not many reports of tests which throw light on these questions, due to limited range in water-ratio, age, etc. Further studies will undoubtedly throw much new light on many features of concrete properties and behavior.

Readers are referred to the JOURNAL *for December 1931 (Vol. 28), for further discussion which may develop. Such discussion should reach the Secretary by Nov. 1, 1931.*

MATERIALS

ADMIXTURES

Bentonite and its use in the industry. *Le Genie Civil* (France), Oct., 1929; reviewed in *L'Industrie Chimique* (France), June, 1930, V. 17, No. 197, p. 407.—Bentonite is colloidal clay, which is found in large deposits in the United States and Canada.* When brought in suspension in water, part of its constituents hydrolyze. It can be used for following purposes: as detersive agent, as stabilizer for oil emulsions, as water softener, especially for permanent hardness, and as concrete admixture. Bentonite forms with great number of oils stable emulsions.—A. E. BEITLICH. (*According to U. S. Bur. of Mines Tech. Paper 438, 1928, Bentonite occurs in quantity in Wyo., Nev., Calif., S. D., Id., N. M., small deposits probably a dozen other states; little or none in states east of Mississippi.—EDITOR)

AGGREGATES

Behavior at high temperatures of concrete containing blast furnace slag. RICHARD GRUN AND HUGO BECKMANN. *Archiv fuer das Eisenhuettenwesen* (Germany), May, 1930, V. 3, No. 11, p. 677-83.—The statement confirms itself that quartzous aggregates are less suitable for concrete which is exposed to high temperatures. It was found that concrete made with blast furnace slag shows least decrease in strength under heat treatment. Slag may be mixed with cement or used as aggregate in concrete. It seems that the best concrete for high fire resistance can be made from dense, strong slag as aggregate by use of the voids-curve or Fuller curve and by use of cement containing blast furnace slag.—A. E. BEITLICH

Corrosion of steel by breeze and clinker concretes. *Dept. Sci. Ind. Res., Special Report* No. 15, (England).—Instances have been reported of serious corrosion of steel in contact with breeze and clinker concretes, which are much used because of low density and cheapness. Tests undertaken to investigate these effects included an examination of behaviour of solutions derived from action of moisture on various types of coal residue aggregates. Since it has been found that corrosion may be rapid even under dry indoor conditions, examination was made of the corrosive power in moist air but out of contact with the sources of liquid moisture. Also tests were made to ascertain relation between permeability and corrosive power, and the effect of replacing fines in breezes and clinker aggregates by sand. It is considered that sulphur content of the materials is prime cause of corrosion and that porosity and permeability are of minor significance in this connection. Accounts are given of the various tests, and the particular properties of coal residue aggregates responsible for corrosion with their relative importance are indicated.—JOHN E. ADAMS

New Pycnometer measures free moisture in aggregates. D. D. McGUIRE and W. H. ROWAN. *Concrete*, May, 1930, V. 36, No. 5, p. 41.—Change in Tennessee highway department specifications for control of proportions of fine and coarse aggregate by weight made it evident that one of major factors of this type of control was accurate knowledge of correct per cent of free moisture in aggregates and their apparent specific gravity. New type pycnometer was developed from several revamped pieces of equipment for this purpose. Container, is made of non-corrosive, metallic material, such that its base is 10 in. in diameter and top 3 in. in diameter. Height is 12 in. Under bottom are placed three round points of contact with weighing table of a solution balance. To determine apparent specific gravity, it is only necessary to determine weight of can and bar, water which can will contain to point where moisture gauge bar touches its surface, and weight of sample under

consideration. To determine free moisture, apparent specific gravity, G_s, in saturated and surface-dried condition, and apparent specific gravity, G_w, in a wet or stock-pile condition, are required. From formula, $\frac{G_s - G_w}{G_s \times (G_w - 1)} \times 100 = M$, the per cent moisture may be readily obtained.—C. BACHMANN

Use and abuse of building materials. A. M. GILES, *Builder* (England), June 6, 1930, and June 13, 1930, V. 138, No. 4557, and No. 4558, p. 1099 and 1149.—Concrete: The historical development of the use of concrete is outlined and reference made to the greater lasting quality of concrete as compared with stone. For aggregate any hard substance may be used, although cinders are not recommended owing to corroding effect on the steel. It is recommended that sand be screened from gravel and remixed to ensure proper proportions. Limestone is liable to decomposition under great heat and is not suitable for fireproof work. Essential quality of all aggregates is cleanness and freedom from earthy matter. This should be tested by allowing sample to stand in water for a time. Dirt will collect on the top of the aggregate as a film. A sample may also be tested by shaking up with weak solution of caustic soda and allowed to stand for 24 hours. If solution above aggregate is brownish yellow too much impurity is present. Broken brick should always be well cleaned and soaked in water before mixing. In parts where washed aggregates are not obtainable, it is often advisable to fit up washing and screening apparatus combined, on site. For non-porous concrete coarse aggregate should pass a ¾ in. ring and be retained on a ⅛ in. mesh. Aggregates: For dense concrete voids must be filled; sand as well as shingle should be well graded. Too much dust in aggregates causes weak concrete since more water is required for mixing. Scientific proportioning and right consistency should be aimed at. Too dry mixes, though strong, are porous, while wet concrete takes long to set and is weaker. Methods of arriving at proper proportioning are outlined, and reference is made to the fineness modulus method of proportioning by analysis.—JOHN E. ADAMS

CEMENT

Determination of the specific gravity of cement. KARL E. DORSCH. *Tonind. Ztg.* (Germany), May 8, 1930, V. 54, No. 37, p. 627-8.—For rapid but accurate determination of specific gravity of cement a large mouth pycnometer with a long graduated, ground-in tube ending in a small funnel is used. One hundred g. cement is introduced into pycnometer which is stoppered and shaken thoroughly. Then more C_6H_6 is introduced from a burette to give an accurate measure of volume after temp. adjustment is made.—F. O. ANDEREGG

Cement. *Contr. Jour.*, June 25, 1930, V. 102, No. 2663, p. 2058.—Presence of coal in breeze aggregates may cause failure in cement renderings on concrete made with them, as well as in the concrete itself. The effect of an excess of sulphides in basic slag aggregates is not marked in dense and impermeable concrete, though its effect is very evident in plain and reinforced concrete. Calcium sulphate combines with calcium aluminate in the cement and forms calcium sulpho-aluminate, and causes disintegration. Swelling and cracking in cement rendering on brick work may be caused by sulphuric anhydride in the bricks. Concrete roads have been known to fail in same way through absorption of this compound from road sub-base composed of stone and slag. Ground waters near coal deposits, tanning liquors, fatty acids, creosote and sugar tend to disintegrate concrete exposed to their action. Whin chips and whin dust result frequently in failure to harden—due perhaps to excessive water required to wet the dust in mixing. River gravel, when coated with algae may not bond well with cement.—JOHN E. ADAMS

Lehigh Valley cement plant operates on America's pioneer mill site. *Pit and Quarry*, May 7, 1930, V. 20, No. 3, p. 36-37, 33 ill. Explains David O. Saylor's portland cement project in 1866, his patent in 1871, and the use of his product more than 50 yrs. ago in constructing the Eads jetties at mouth of Mississippi River. Coplay Portland Cement Co. is direct successor, occupies original site, and quarries from same deposit. At this plant can be found the old fashioned, dome-type lime kiln, high vertical kiln, and improved rotary kiln. Originally rock and clinker were ground with buhr stones. This is the first cement plant to be electrically operated. Limestone deposit is about 6 mi. north of Allentown, near Coplay, Pa. It produces almost pure calcium carbonate and an argillaceous stone, each kind running uniform in color and quality. Quarry has been worked 25 yr. The face is 180 ft. high and the floor has an area of about 7 acres. Lower levels are being started. On a jig-back cars of stone are hauled up 400-ft. incline, one at a time, by an electric hoist. Primary gyratory crusher reduces the stone to 3-in. This stone is hauled in trains of 12 cars to main plant by a 16-ton gasoline locomotive. Cars are dumped by hand over a rectangular concrete 2,000-ton bin. Following 54-in. by 24-in. roll crusher, flow sheet includes magnetic separators, pulverized-coal-fired rotary dryers, automatic proportioning scales, a hammer mill, 18 fine-grinding mills, storage and blending tanks, 4 steel kiln-feed tanks and four 125-ft. kilns using pulverized coal as fuel.—A. J. HOSKIN

A thermal study of the manufacture of cement. H. LEFORT. *Rev. materiaux construction trav. publics.* (France) 1930. No. 248, p. 171-6.—Another heat balance calculation.—F. O. ANDEREGG

The properties of the cement bacillus and its development during disintegration of concrete in sulfate waters. A. GUTTMANN and F. GILLE. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 46, p. 759-62.—There can be no doubt about existence of calcium aluminum sulfate as the cement "bacillus." The best protection consists in making a dense and rich concrete. —F. O. ANDEREGG

Viscosity of setting cement. Immersion filter method. K. E. DORSCH and A. DEUBEL. *Kolloid Zeitschrift* (Germany), April, 1930, V. 51, No. 1, p. 180-6.—Investigations were carried out at Institute of Technology in Karlsruhe (Germany) concerning viscosity of cement mixed with water for normal consistency. Tests made with usual method did not give reliable results on account of sedimentation. New immersion filter method was derived, which allows measurements of viscosity of cement during period of setting with accuracy of 5 per cent. Progress of setting of a number of different cements was studied. During the first time of setting (0-60 min.) viscosity increases rapidly. This period is followed by constant viscosity for ½ hour. After this time viscosity increases again. Results of experiments agree with theories of setting.—A. E. BEITLICH

Annual meeting of German Portland Cement Manufacturers Association, March, 1930. *Zeitschr. angew. Chem.* (Germany), May, 1930, V. 43, p. 355.—The following papers were presented: (1) Gustav Haegermann: Sintering of cement raw materials. Rate of combination in burning cannot be judged from appearance or shrinkage of clinker. Best temperature for sintering of cement raw materials is between 1275° and 1295° C. (2) Eitel: Hydrates of tricalcium-aluminate. Thorvaldson's results on disintegration of cement by sulphate waters are discussed. (3) W. A. Roth: Thermochemistry of cement. Heat of reaction, which is formed when cement raw mix is burned with paraffin and compressed oxygen in a calorimeter, was calculated. Investigations were made to study heat of formation and heat of setting by dissolving cement raw mixes in hydrofluoric and hydrochloric acid. (4) Hans Kuehl: Relation between strength of cement and fineness. Tests indicated that strength of cement does not increase appreciably when particle size drops

below 20 microns. Suggestions are given for manufacture of high strength cement by certain combinations of particle sizes. (5) F. Krauss: Importance of complex compounds. (6) H. Burchartz: Effect of storage on cement. Discussion of the theory by Gensbaur. (7) R. Nacken: Heat of formation of silicates. Heat of solution was determined by dissolving in mixtures of hydrochloric, hydrofluoric and nitric acid. (8) L. Forsen: Gypsum and other retarding agents. Theory of setting and hydration of cement is presented. (9) Otto Graf: Concrete disintegration in aggressive waters. Paper discusses resistance of different kinds of cements against aggressive waters. Influence of fineness is cited and suggestions for concrete protection are given. (10) A. Hummel: Fineness modulus of aggregates. (11) A. B. Heller: Improvements in mill practice. New type of mill for closed circuit grinding is described.—A. E. BEITLICH

The first Dutch cement works, at Saint Pietersberg, near Maestricht. *Revue Materiaux Construction trav. publics* (France), Jan., 1930, No. 244, p. 19-23.—This is a works for treatment of chalky marl obtained at Saint Pietersberg, and turns out annually 300,000 tons of cement. (1,000 tons of clinker per diem). The chalky marl is treated together with imported plastic clay (levigation). The powdered material and the slime are transported with the aid of pneumatic apparatus. All the most up to date implements are used for crushing, grinding and mixing process. Coal is also dried and treated with success.—A. SIZE

Enlargement of the Portland Cement Works at Groschowitz (Upper Silesia). *Revue Materiaux Construction trav. publics* (France), Feb., 1930, No.245, p. 74-76. A description of alterations to cement works in Upper Silesia (manufacturing artificial portland cement by the wet process) to increase production from 200 tons to 500 tons per day. Works are easily controlled and output very regular. Nearly all the machinery is operated individually. —A. SIZE

Suggestions for selection of lining for cement kiln, especially for rotary kiln. OTTO FREY. *Zement* (Germany), May, 1930, V. 19, No. 21, p. 492-3.—In selecting suitable linings for cement kilns it is necessary to make chemical analysis of raw material and firebrick. Clinkering point and melting point must be determined. Clinkering point of lining must be more than 50° C. higher than clinkering point of raw mixture.—A. E. BEITLICH

The rotary kiln and waste heat boiler. S. STEINER. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 52, p. 858-60. A review of the arguments for and against the waste heat boiler in connection with the rotary kiln.—F. O. ANDEREGG

Portland cement plant "Kursachsen" in Carlsdorf (Germany). *Zement* (Germany), May, 1930, V. 19, No. 20, p. 464-7.—Plant was built in 1928 in Carlsdorf a. U. near Naumburg a. S. and was enlarged in summer 1929. Output is 500 t. daily. Location of plant is favorable with water way connections to northern and central parts of Germany. Raw materials are limestone and clay. Quarry is situated 500 m. from the plant. Stone is transported by aerial tramway to crusher; discharge of cars is automatic. Belt conveyer delivers crushed materials to raw mill, where both raw materials are stored in bins. Two table feeders at bottom of bins control mixture. Both feeders are driven by same motor, speed of which can be regulated. Raw mixture and water enter two three-compartment-mills. Two special bins with table feeders are installed to add admixtures for manufacture of special cements. Slurry runs to four concrete tanks with conical bottoms. Elevator buckets transport slurry from outlet of tanks to distribution arrangement on top. Compressed air provides good mixing. From there raw mixture is delivered to kiln feeders which are regulated by individual motors. Plant has two Unax kilns with coolers. Two finishing mills, each one having four chambers, are used for cement grinding. Screws and elevators bring finished cement to four storage

bins of reinforced concrete. These are provided with flat bottoms, and cement is moved from bins to packers by pumping system. Latter enables mixing cement from different bins. Coal arrives at plant in carloads and is fed directly to crushers. Conveyer belt carries material to two coal dryers and storage bins. Special consideration is given to perfect dust precipitation. Power of 30,000 volts is furnished to plant from outside and is transformed to 500 volts in own power station. Plant equipment and machinery were installed by F. L. Smidth & Co. A/S, Kopenhagen and Luebeck. Several photographs accompany article.—A. E. BEITLICH

Cement chemistry in theory and practice. "Zementchemie in Theorie und Praxis." HANS KUEHL. 1929. Editor: *Zement und Beton*, G. M. B. H., Berlin, 92 pages. R. M. 4.50. Reviewed in *Zement* (Germany), May, 1930, V. 19, No. 18, p. 432.—Subjects of book are: ways of cement research, theory of cement burning, constitution of clinker, hardening problem and chemistry and technique of high early strength cements. Book is of great value to cement research workers and cement manufacturers.—A. E. BEITLICH

Heat of mill and setting time of cement. K. KOYANAGI. *Zement* (Germany), May, 1930, V. 19, No. 20, p. 467-9.—Investigations were carried out in Japan with gypsum which was heated at temperatures from 130-200° C. for half an hour. Content of SO_3 and loss on ignition were determined. Treated gypsum was mixed with clinker in proportions from 0.9-2.5 per cent SO_3. Setting tests showed that setting time decreases with increasing temperature of heating gypsum. When SO_3 content is lower than 1.2 per cent, cement never shows quick set. Difficulties were overcome by keeping SO_3 content between 0.9 and 1.2 per cent. (cf. *Zement* (Germany), Nov., 1929, V. 18, p. 1348-50; *Journal Amer. Concrete Inst.*, Jan., 1930, p. 3.)—A. E. BEITLICH

Rotary kiln calculation. A. B. HELBIG. *Zement* (Germany), May, 1930, V. 19, No. 19, p. 440-4.—Author gives detailed calculation of heat economy of rotary kiln and discusses following points: heat consumption for burning off carbon dioxide of raw mixture, for vaporization of water, heat loss due to conduction and radiation in kiln and cooler and heat of formation of clinker. Utilization of rotary kiln waste gases can be made very economical when difficulties caused by dust are overcome. Automatic recording thermometers and carbon dioxide meters should be installed on every kiln. Conditions are cited which lead to wrong registrations on those apparatus.—A. E. BEITLICH

Power and heat economy in cement industry. Directions for examination and valuation of kilns and burning processes. JAKOB KOCH. *Zement* (Germany), April, 1930, V. 19, No. 14, 15, 16 and 18, p. 324-8, 348-53, 373-7 and 418-23.—Burning process in cement kiln consists of number of separate processes. Author illustrates operating conditions by means of graphs. Dry and wet processes are discussed. Formulas are given for calculation of heat consumption, amount, temperature and heat content of waste gases and their dependence on the operating conditions. Relations are shown between amount of waste gases, heat consumption and excess of air and suggestions are made for economic utilization of waste gases. Efficiency of waste heat boiler system is calculated for dry and wet process, and influence of vapor pressure on economy is illustrated. Several tables show amount of waste gases and their temperature in both processes by different heat consumption and also necessary amount of heat for vaporization of water dependent upon water content in raw material. One single diagram is given which shows amount of gases for fuels with any calorific value, any coal consumption and any excess of air. Same diagram enables one to read heat content of waste gases, their temperature and preheating temperature of combustion air for any heat consumption for burning of dry raw mix, for any water content in raw mix and for any loss in heat by radiation and in clinker. A number of examples of practice are given which illustrate the use of these diagrams.—A. E. BEITLICH

Importance and evolution of the concrete industry in Italy. ANTONIO PESENTI. *L'Industria Italiana Del Cemento* (Italy), Dec., 1929, p. 4-5.—Cement industry started in Italy at about same time as in United States and at first proceeded on an empirical basis with correspondingly poor product. Wars and political vicissitudes retarded development, despite presence of abundant raw material, until 1870 when national unity was achieved. First plant for production of artificial portland cement was built in 1873, followed rapidly by numerous plants in Casalese, Bergamasco, Veveto, Emilia, Toscana, Umbria and elsewhere. In 1904 discovery of rich beds of marl led also to erection of plants along the Adriatic coastline. In 1898 and 1904 there were severe depressions but in 1904 began a period of prosperity which ended with the World War. Old plants were enlarged and new plants constructed and scientific methods were introduced. After the war there was great expansion of cement industry so that it now is one of most important industries of country representing a capital employed of 1,100,000,000 lire. Production increased until 1927, since which time it has remained practically stationary. Manufacturers however, have continued to improve their plants and processes and Italia cements are equal to the best foreign products, including special early hardening cements. The Federazione Nazionale Fascista del Cemento and the Societa' Incremento Applicazioni Cemento are recently formed organizations to encourage improvement in manufacture and to promote use of cement.—C. G. CLAIR AND M. N. CLAIR

Rotary kiln calculations. HANS BUSSMEYER. *Zement* (Germany), May, 1930, V. 19, No. 22, p. 522-4.—Author criticizes article by A. B. Helbig and contests his statement that normal rotary kiln with waste heat recovery is superior to "Lepol"-kiln manufactured by Polysius, Dessau (Germany). Several calculations of Helbig are corrected and tables recalculated. It is claimed that the "Lepol"-kiln has several advantages over any other rotary kiln. Heat calculations for both kinds are given and heat economy of "Lepol"-kiln is especially characterized. (c. f. A. B. Helbig, *Zement* (Germany), May, 1930, V. 19, No. 19, p. 440-4.)—A. E. BEITLICH

Use and abuse of building materials. Cement. A. M. GILES. *Builder* (England), May 2, 1930, V. 138, No. 4552, p. 870; No. 4553, p. 928; No. 4554, p. 969; No. 4556, p. 1013.—Article opens with brief history of development of cement manufacture and its composition. Cement contains 60-65 per cent lime, 20-25 per cent silica, 6-10 per cent alumina, small percentage of oxides of iron, and smaller percentages of other impurities. In setting, it is assumed that silicate of lime breaks down from 3 $CaO.Si\ O_2$ to a compound having one molecule of lime, the other molecules combining with water to form hydrate of lime. Setting time may be delayed by combination of sulphur anhydride or SO_3 with lime to form lime sulphate. Therefore the percentage of sulphur anhydride should not exceed 2.75 per cent. Ratio of lime (after amount to satisfy sulphur anhydride has been deducted) to sum of silica and alumina content should not be greater than 2.9 or less than 2. Manufacture. Main points of interest after excavation of raw material are crushing and grinding, and in wet process, final adjustments as to composition of slurry. Slurry passes down rotary kilns and its temperature rises to 3000° F, causing it to be first dried and then calcined. Resulting clinker is ground to finished cement. Samples for test are required by British Standard Specification to be taken from quantities of not more than 250 tons. The main tests required are for fineness of grinding, setting time, chemical composition, tensile strength of neat cement, and of mixture of cement and sand, and finally soundness. In addition to usual tests for setting time, rough method suggested by Dr. Faber in the Concrete Year Book is described. A 3-in. square pat, 1 in. thick should be placed on a piece of iron, slate or glass. Its progress of setting should be observed and in 18 to 24 hrs. it should have hardened sufficiently to resist any

impression by thumb-nail, while in 48 hours it should not be easy to break pat with fingers. Although there is considerable decrease in volume when dry ingredients of concrete are mixed with water, when concrete has set there is expansion which may not show for several years. Soundness test gives a measure of effect of this post-setting expansion. Expansion comparable with that resulting from long period may be obtained in a few hours by placing the cement in boiling water. The sample should be aerated by being spread out in a layer not more than 3 in. thick and left for 24 hours in a temperature of 58 to 64° F. The hardness test is well known as the "Le Chatier" test. A simpler method consists of placing a 3-in. diameter pat on a 4-in. sq. plate of glass. This is kept in moist atmosphere for 24 hours and then placed an inch above boiling water for about 5 hours. After cooling it can be removed from the glass, and should have retained its lens shape and give a clear sounding ring when struck with knuckle or lead pencil. Cement with high alumina content is quick setting and although, on account of its composition it does not conform to the standard specification, its strength is quite up to standard. Much heat is evolved during setting—an advantage in frosty weather—but it should be kept wet in order to make up for the water lost by evaporation. Care must be taken to avoid contamination by lime or ordinary portland cement. For white cement, iron oxide must be absent.—JOHN E. ADAMS

Hardening of Sorel cements. A. KRIEGER. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 36, p. 577-8.—The compound, $2MgO.MgCl_2.9H_2O$ is always present, but other substances in a gel form are also there for ratio of $MgCl_2$:$Mg(OH_2)$ may vary from 1:3 to 1:15. It is suggested, however, that initial hardening is due to this compound. Strength seems to depend upon the water-cement ratio.—F. O. ANDEREGG

Gaize cement for sea work. G. BAIRE. *Rev. materiaux construction trav. publics* (France), 1930, No. 248, p. 168-70.—Gaize is a siliceous material of sedimentary origin which is about ⅓ soluble in 20 per cent KOH after 20 hours of cold contact and 4 at 50 to 65° C. Exposure to sea water seems to increase the amount of active silica. A good cement is obtained on grinding one part of gaize with two of portland cement clinker plus a trace of gypsum. Its use was interrupted by the war, but certain experimental installations in sea water in 1910-11 have stood up so well that its use in sea water has been revived.—F. O. ANDEREGG

Blast furnace cements. A. O. PURDON. *Rev. materiaux construction trav. publics* (France), 1930, No. 249, p. 205-12.—Blast furnace cements, made by grinding portland cement clinker with blast furnace slags of proper composition, are giving very good results. Early strengths are lower than those of portland cement but improve later. Volume change is small; resistance to salt solutions is good; while danger from sulfur in slag is regarded as negligible.—F. O. ANDEREGG

Researches on action of inhibitors of setting of cements. *Rev. materiaux construction trav. publics* (France), Feb., 1930, No. 245, p. 50.—Berger gives following conclusions from researches carried out with 4 samples of anhydrite, one sample of gypsum, one of plaster and cement clinkers. To obtain cement which sets normally with anhydrite, it is necessary to use more SO_3 than with gypsum or plaster of Paris. The plasticity of cement which contains anhydrite is less than that of a cement in which gypsum is used. The fineness of the anhydrite has considerable importance with regard to plasticity of cement, but various qualities of natural anhydrite do not present any distinctive features. It is preferable to crush slow setting component separately from clinker. Quantity of SO_3 existing in solution of "inhibited" cement is

proportionate to solubility of inhibitor used. This aspect of subject deserves further study as results obtained so far are not final.—A. SIZE

Dosage of soluble silica in pozzuolanic cements. BAIRE. *Revue materiaux construction trav. publics* (France), Feb., 1930, No. 245, p. 61-62.—Method indicated by Chesneau is only suitable for estimating soluble silica in unmixed cements; for pozzuolanic cements the results obtained are not satisfactory, any more than in regard to cements obtained by mixing pozzuolane with portland cement. Author recommends combining hydrochloric acid method and carbonate of soda method for mixed cements, as ensuring complete dissolution. The mixtures contain at the same time silicates and non-combined silica.—A. SIZE

Hydrates of tricalcium aluminate. WILHELM EITEL. *Zement* (Germany), April, 1930, V. 19, No. 17, p. 394-9.—Discussion of research work on calcium-aluminates in connection with work done by Th. Thorvaldson. Studies concerned the volume changes of test pieces in sulphate waters. Hydrates of the tricalcium-aluminate were examined and reactions with water studied. Four different hydrates could be identified and their optical properties determined. These are hydrates with 6, 8, 10½ and 12 molecules of water. The heat of solution of these compounds is cited. A great number of references are given.—A. E. BEITLICH

MISCELLANEOUS

Report on accident during application of protective coating on concrete. DR. RICHARD GRUN. *Zement* (Germany), April, 1930, V. 19, No. 16, p. 383-4.—A water tank with a capacity of 100 cu. meters was provided with a protective inner coating of a bituminous preparation. Evaporating solvents formed with air explosive mixtures which caused death of two laborers. Following precautions should be observed: coating material should not contain any substances with low boiling point; flash point should be determined, smoking and open lights must be prohibited during and after application of coating. Sufficient ventilation must be provided for in closed chambers and tanks.—A. E. BEITLICH

Contribution to knowledge of old mortars. KARL BIEHL. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 51, p. 839-41; No. 53, p. 877-8.—Samples of mortar from German buildings ranging from the 16th to 19th centuries have been studied. Most of them were quite fat. Trass was found in only one mortar and then in small amount. From Persia one mortar was of gypsum without sand, while one of later date was fat lime mortar containing ground brick, showing influence of Romans.—F. O. ANDEREGG

Magnesia cements. *La Revue Industrielle* (France, Feb., 1930; reviewed in *L'Industrie Chimique* (France), June, 1930, V. 17, No. 197, p. 406.—Most important magnesia cement is "Sorel"-cement, which consists of mixture of magnesium oxide and magnesium chloride. Variations in cement are due to composition of magnesium oxide, which is obtained by calcination of magnesite or magnesium carbonate. Good magnesite, which is used for very pure cements, is calcined at 500° C., while magnesites containing ferric oxides need a temperature of 800° C. Ignited material is mixed with different materials such as saw dust, asbestos fibres, sand, cork or diatomaceous earth. These mixtures so obtained are mixed with solution of magnesium chloride of 22° Be. Author describes also some other methods of manufacture of magnesia cements. A. E. BEITLICH

Refractory mortars. O. PHILIPP. *Chem. Fabrik* (Germany), 1930, p. 86; reviewed in *L'Industrie Chimique* (France), June, 1930, V. 17, No. 197, p. 405.—Refractory mortar, which is to be used for construction of firebrick linings, must have sufficient adhesive properties and should not have a higher porosity after exposure to fire than the bricks. First condition is fulfilled when fire clay is used with or without binder like sodium silicate or dextrin. Refractory mortars, which have to stand very high temperatures are mostly made from the brick material itself by grinding and mixing powder with suitable cement like bentonite. Instead of cement one can use mixture of starch solution and glass or quartz powder.—A. E. BEITLICH

Bauxite production in France. B. CHARRIN. *Le Ciment*, May, 1930, p. 190.—Bauxite deposits are located in Southern France. Of these the Var yielded about 520,000 tons in 1928 and the Hérault 110,000 tons. France leads in production of bauxite with an annual output of over 600,000 tons. United States production, highest in 1920, has dropped to 300,000 tons (5th place) due to inferior quality and higher cost of production. Fifty per cent of bauxite produced is utilized in aluminum industry; 30 per cent is consumed by alumina cements; the rest used in the chemical industries. The world's production of aluminum demands about 850,000 tons of bauxite a year. A conservative estimate of the bauxite still available for exploitation in France is as follows: Var with 8 million tons; Hérault with 5 million tons; Bouches-du-Rhone with 2 million tons and Ariege with one million tons. Half of the above deposits contain a maximum of 7 to 8 per cent silica, a limit acceptable in the manufacture of aluminum and alumina cement.—MRS. M. A. CORBIN

Chemical composition of bauxites. V. CHARRIN. *Génie Civil* (France), Jan. 18, 1930, V. 18, No. 3, p. 63-64.—Red bauxites which are very abundant in France, have just begun to find very wide application in connection with cement. Alumina cements require a mineral not surpassing 8 to 10 per cent of silica and containing 30 per cent of iron and a proportion of 50 per cent at least of alumina. France has most and best quality bauxites, but in view of the ever-increasing demand for them for making aluminium and alumina cements it would be necessary to regulate the industry, and to amend present laws with regard to quarrying (law of 1810).—A. SIZE

Hydration of calcined gypsum. W. C. HANSEN, *Ind. Eng. Chem.*, June, 1930, V. 22, No. 6, p. 611-613.—A study was made to supply satisfactory explanation of accelerating or retarding action of soluble substances upon setting of gypsum-water pastes. Time-temperature curves obtained with calcined gypsum in various salt solutions indicated that controlling factor in setting is rate at which precipitation of gypsum takes place from its supersaturated solution. This theory was corroborated by demonstrating that foreign salts have a marked effect upon rate of gypsum precipitation from mixed solutions of calcium nitrate and ammonium sulfate. It is possible that effects of foreign substances on setting of portland cement also may be explained by their effects upon rate of precipitation.—ROY N. YOUNG

PROPERTIES OF CONCRETE

Trouble of concrete. *Australasian Eng.*, May 7, 1930, V. 30, No. 167, p. 20.—Contraction of air hardened concrete is about three times greater than that of concrete hardened under water. Sulphur in aggregate may also cause excessive expansion. An example is given of 3-in. concrete floor which pushed a wall over 2 in. in six months. Excessive heat may cause disintegration; a concrete foundation to a furnace, separated from the actual fire by 2 courses of 9 in. brickwork could, after a time, be broken away with the fingers. Broken firebrick aggregates are satisfactory if ample time is allowed for maturing and hardening delayed by placing wet bags over concrete. Finishing surfaces of

floors should be placed while under layer is still green, or there will not be proper cohesion between the two layers.—JOHN E. ADAMS

New comparisons between tensile, flexural and compressive strengths. R. FERET. *Rev. materiaux construction trav. publics* (France), 1930, No. 244, p. 1-4; No. 245, p. 52-60; No. 246, p. 83-95; No. 247, p. 127-33. —Search of literature has brought to light several comparisons of tensile, flexural and compressive strengths. Results are plotted and in general it is found that first two properties are proportional to each other and to the $\frac{2}{3}$ power of compressive strengths. Some exceptions exist, however. On plotting the ratio between the flexural and compressive strengths against water-cement ratio a number of straight lines were found. In general, alumina cements have lowest tensile-compressive ratios, followed by the portland cements which set lime free, and then the pozzuolanic cements. With moist storages and within certain limits of aggregate compositions, the water-cement ratio is found to be most important in controlling the strengths. On storing in corrosive waters, like $MgSO_4$, results vary with nature of cement.—F. O. ANDEREGG

The impermeability of concrete to water. C. R. PLATZMANN. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 40, p. 676-7; No. 43, p. 718-20; No. 45, p. 748-9.—After review of literature, importance of grading of aggregate is pointed out. Various methods of surface hardening and surface protective coatings are discussed.—F. O. ANDEREGG

Use of concrete and reinforced concrete on the railways. M. F. B. FREEMAN AND M. E. KRICK. *Génie Civil* (France), Feb. 8, 1930, V. 96, No. 6, p. 144.—Concrete and reinforced concrete are widely used for various railway undertakings owing to low cost of upkeep and their refractory character. But it is necessary to keep a close watch on changes caused by time, etc., and to take precautions in regard to calculation of weights to be supported by bridges. Smoke of locomotives has no deleterious influence on concrete provided surface is smooth and has no fissures. Iron and steel bars must be sufficiently far from surface of concrete to avoid attack by rust.—A. SIZE

Theory of the expansion of liquid cements. HENRI LAFUMA, *Rev. Mat. Constr.*, Jan., 1930, No. 244, p. 4-8.—Swelling of quick lime by slaking is due to the spontaneous pulverization of the lime and increase of absolute volume of solid phases during reaction. Apparent volume is enormously increased by this process. It is necessary, in order to improve mortars, to eliminate these reactions by dissolving solid salts beforehand. Author studies successively chemical disaggregation of aluminous and hyper-aluminous portland cements; setting of tempering cements; swelling action which is exercised on mortar by presence of aluminium plates, and the expansion of quick setting mortars. —A. SIZE

Concrete; effect of aggressive liquids on concrete and possibilities of concrete protection. M. GRUEN. *Chimie et Industrie* (France), Jan., 1930; reviewed in *L'Industrie Chimique* (France), June 1930, V. 17, No. 197, p. 406.—In general all acids attack concrete. This effect increases with concentration of acid. Gaseous acids too have corrosive properties. Certain salts are able to attack concrete. Sulphates have the tendency to form calcium-sulphoaluminates. Fatty substances corrode concrete. In order to prevent concrete attacks, one can use suitable mixtures, or protect surface with coatings. Grain size of sand plays important role. Coating may be either bitumen, tar or some varnish.—A. E. BEITLICH

Contribution to valuation of certain mixed storage of cement test pieces for practice. H. BURCHARTZ. *Zement* (Germany), May, 1930, V. 19, No. 21 and 22, p. 488-92, 510-5.—Investigations were made of variations in strength of test pieces under different storage conditions. Five different kinds of cement: one high-early strength cement, one portland cement burned in rotary kiln, one portland cement burned in shaft kiln, one iron-portland cement and one blast furnace cement were studied. Normal strength tests were made

under varying storage conditions, and gain or loss of water during storage was determined from each test piece. Strength of test pieces which are stored in water after preceding normal storage decreases in general. Tensile strength is more effected than compressive strength, especially during short water storage. —A. E. BEITLICH

Effects on concrete. "Einfluesse auf Beton." A. KLEINLOGEL, F. HUNDESHAGEN AND OTTO GRAF. 3rd edition, 1930, Wilhelm Ernst & Sohn, Berlin (Germany), R. M. 39.50; reviewed in *Zement* (Germany), May, 1930, V. 19, No. 19, p. 456.—Book illustrates possibilities of corrosion of concrete, danger and damage through chemical agents, aggressive waters and other unfavorable conditions and shows ways to prevent such damage by proper precautions. Many references are given.—A. E. BEITLICH

Building protection. "Der Bautenschutz." (New journal) Wilhelm Ernst & Sohn, Berlin (Germany); reviewed in *Zement* (Germany), May, 1930, V. 19, No. 18, p. 432.—Journal appears together with *Beton und Eisen* (Germany), and brings articles concerning new tests, investigations and experiences in the field of concrete and effect of water, weather and fire on concrete.—A. E. BEITLICH

Equipment for testing the water impermeability of concrete. HANS HECHT. *Zement* (Germany), May, 1930, V. 19, No. 20, p. 472-3.—Author describes apparatus similar to one designed by Otzen. Water pressure is kept constant by means of compressed air tank which is connected with large water reservoir. Twelve test pieces can be exposed simultaneously to different pressures. Apparatus can also be used for tests with aggressive waters and solutions and is entirely independent of any stationary water line. Test pieces up to 30 cm. in diameter can be tested. Pressure can be varied up to 20 atm.—A. E. BEITLICH

Concrete and reinforced concrete in fires. New experiences. A. WEISS. *Zement* (Germany), May, 1930, V. 19, No. 18, p. 424-7.—Behavior of concrete walls and ceilings during recent fires showed again superiority of concrete for fire resistance. Casings of iron columns should not be thinner than 10 cm.—A. E. BEITLICH

Control practice in reinforced concrete construction in erection of warehouse No. VII in Free Port of Stettin. (See FIELD CONSTRUCTION - Buildings.)

Blast furnace cements. (See MATERIALS—Cement.)

ENGINEERING DESIGN

Welded trusses developed for bridge floor reinforcing. H. M. BRINGHURST. *Eng. New Record*, July 3, 1930, V. 105, No. 1, p. 8.—Roadway slab reinforcing used on the new bridge over the Ohio River at Louisville, Ky., and on the Ambassador Bridge at Detroit is interesting example of truss-type reinforcement, and represents first use of arc welded material on bridges of this size. Reinforcing units are light trusses of Warren type made up of round bars. Web is formed by a continuous bar bent to a zig-zag shape and welded at bends to top and bottom chords, each of which consists of two straight round bars. Bars from which trusses are fabricated are furnished by mill in coils and are straightened at plant in straightening machines to remove mill scale. Webs are bent on compressed air machine and are spliced to facilitate handling. The bars are then assembled in a jig and each joint is electrically arc welded by hand. Special care is taken to avoid distortion from application of heat. After removal, each joint is tested with a pinch bar. At irregular intervals, trusses are tested to destruction as additional check. Specimens of long span fail from buckling of the top chord and those of short span fail by buckling of the web. Weld failures were not obtained in any tests.—D. E. LARSON

Design of road bridges. C. S. CHETTOE. *Roads and Rd. Constr.* (England), May 1, 1930, V. 8, No. 89, p. 149.—Rigid Frames. This article continues with discussion of rigid frames of semi-elliptical and parabolic type of arch frames which meet their abutment slabs at an almost vertical angle. They are more economical in material than true rigid rectangular frames and have better appearance. Attention is drawn to difference between active pressure and passive pressure exerted by earth in case of retaining walls, the latter being, when maximum, much greater than former. According to Rankine, maximum passive horizontal pressure bears same ratio to downward pressure at any point, as downward pressure bears to active horizontal pressure. Although assumption that pressure is horizontal may hold when walls are constructed in timbered trench against vertical surface of consolidated earth, it is not true when filling is piled behind wall. In general, it is sufficient to design for active pressure. Passive pressure on frame sides reduced crown stress but will increase moments at corners. For very flexible frames it may be advisable to allow for some passive pressure. In culvert design, the more flexible the culvert the less weight of earth carried by it. Conditions will be more favorable if culvert is excavated out of existing ground up to haunches. Increased side pressure due to steep sides to a ravine has occasionally caused failure of culverts constructed in it. Filling should be brought up equally on both sides, otherwise allowance must be made in design. Reference is made to Kleinlögel's book for cases of side loadings.—JOHN E. ADAMS

A new department store in Frankfurt a. M. (Germany). RUDOLF KLAR. *Zement* (Germany), May, 1930, V. 19, No. 18, p. 431-2.—Building is located in business center and limited space was available for transportation and storage of construction materials. Foundations were placed on bad underground which allowed only 2 kg./cm.^{2} pressure. High early strength cement was used. Skeleton structure shows beams with span of 8 meters. A total of 8500 m^2 ceilings were placed. Ceiling of basement is reinforced concrete, one over first floor is crosswise reinforced Remy ceiling, rest are ordinary reinforced Remy ceilings. Cross sections of beams were made as small as possible for architectonical reasons and their reinforcements were made especially strong. Concrete test pieces gave 280 kg./cm^2. compressive strength after 12 days.—A. E. BEITLICH

Specifications of the German committee for reinforced concrete of September, 1925. "Bestimmungen des deutschen Ausschusses fuer Eisenbetom vom September 1925." 5. edition, 1930, Wilhelm Ernst & Sohn, Berlin (Germany), R. M. 1.80. Reviewed in *Zement* (Germany), May, 1930, V. 19, No. 20, p. 480.—Four parts deal with following subjects: specifications for reinforced concrete, concrete ceilings, concrete buildings and cube tests.—A. E. BEITLICH

Modern skeleton building in Kiel (Germany). WENZEL. *Zement* (Germany), April, 1930, V. 19, No. 16, p. 378-9.—Building is 112 m. long and is made without any shrinkage joints. Concrete was kept wet for 6 weeks by means of special sprinkler system and no visible shrinkage cracks could be observed after one year. Floor slabs, which have solid rib in center, have span of 6 m. and are 1.30 m. wide. They rest on 20 x 40 cm. concrete columns. Main floor consists of 14 cm. thick concrete slabs which are supported by 23 cm. thick continuous brick wall.—A. E. BEITLICH

The convention hall in Memmingen (Germany). WECHS. *Zement* (Germany), May, 1930, V. 19, No. 22, p. 521-2.—Description of construction of large hall in reinforced concrete. Building is 66 m. long, 26.50 m. wide and 12.50 m. high. Concrete columns have cross-shaped cross-section; they are 12 m. high and 1.05 and 1.50 m. wide. Side walls are solid and 25 cm. thick.—A. E. BEITLICH

Project of the dam of Tecamachalco. ADOLFO ORIVE ALBA. *Ingenieria* (Mexico), Feb., 1930, p. 60-61.—Results of an investigation made for Secretary of Communications to determine the location and design of dam to control Los Morales river are given. A survey was made to determine the underground conditions. A cross section of the site is given showing geological formation. The conglomeration clay formation near surface in some parts is said to be similar to that under right side of the St. Francis dam, which recently failed; so special precautions were considered necessary. Rubble or rock-fill type of dam was considered most adaptable to the conditions; but not enough materials were available; so gravity and arch-gravity dams of concrete were investigated. It was decided to build a gravity dam, designed assuming a pressure on the base varying from full head of water at upstream face and into line of vertical drains to zero at downstream face. The result of all forces was made to fall within middle third for both empty and full conditions. The coefficient of sliding was kept less than 0.70. A sketch of proposed cross section is given, which shows all important details. Section is of typical gravity kind of cyclopean concrete, four meters wide at top, 16 meters wide at the base, and 23 meters high. There is a series of vertical drains 10 centimeters in diameter spaced at two-meter intervals parallel to upstream face, and two meters from it. Drains are intercepted by an inspection and drainage gallery. Cut-off walls extend below the base of dam both at upstream and downstream sides. Purpose of permeable downstream toe being to prevent the movement of material if it becomes softened by the flow of water. A spillway opening is provided to take care of both the San Joaquin and the Tecamachalco excess flows. An emergency conduit 0.75 meters in diameter provides for emergency flows.—MILES N. CLAIR AND C. G. CLAIR

Design and economy of plain and reinforced concrete dams. E. PROBST AND F. TOLKE. *Zeitschrift Ver. deu. Ing.* (Germany), 1930, V. 74, Nos. 13 and 15.—Paper, presented at Second International Power Conference at Berlin, is brief review of various types of dams and problems encountered in their use. Dam cost is such large part of total cost of a hydroelectric project (29, 29 and 40% for three cases cited) that selection of most economical structure is important. Arched gravity dam is not recommended unless joints are to be grouted. Grouting is advised in spring when masonry is at its lowest temperature. Danger of uplift should not be underestimated. Drainage system will relieve uplift and may be considered as an additional precaution. It should not be considered in design, however, as its continued functioning throughout long life of dam is not certain. Rather arbitrary assumptions of gravity dam proportions are justifiable when uncertainty of stresses near base and force of uplift are considered. Base to height ratios used in recent practice (ten dams listed) indicate that while about 0.70 is used in United States and in Germany, more conservative values (up to 0.97) are common in other European countries. Larger section is not necessarily safer section, because of cracks from higher "secondary" stresses and from shrinkage. Wherever possible arch dam should be used rather than gravity dam. Some American and European types of slab and buttress dams are discussed from viewpoint of cost and durability. These types, compared to gravity and arch dams have advantage of clearly understood structural action. Thin reinforced sections, however, face possibility of corrosion of steel. Condition of foundation rock is of primary importance to safety of structure and should be thoroughly investigated. Proper care is not always used to obtain concrete that resists soakage and frost action. These properties may sometimes be more important than a high compressive strength. Large model tests and observations on existing structures are recommended for increasing knowledge of dam construction. Formation of an international bureau of records on dams is desirable.—A. A. BRIELMAIER

Concrete and reinforced concrete for water structures. F. HIRT. *Zement* (Germany), May, 1930, V. 19, No. 20, p. 478-80.—Draining of large district in German lowlands and necessary concrete structures are described. Total area to be drained was 40 million square meters. Deepest part lies 1.43 m. under sea level. It was necessary to move 500,000 cu. meters ground. Channel was dug 15 m. wide at top and 10 m. at bottom, with fall of 1 cm. per 100 m. Highway bridge of reinforced concrete was built over channel. In order to lead water underneath a second channel a sewer was built of reinforced concrete. Sewer has three openings of 10 m^2 each. Total length is 31.50 m. Underground was soft sand. Third outstanding structure is pumping station near Borssum. Each of three rotary pumps delivers 8 cu. meters water per second and each is driven by two motors, one with 340 H. P. for pumping up to 2 m. and one with 600 H. P. for pumping from 2 to 6 m. Building rests on 400 wooden piles, each having diameter of 28 to 30 cm. Floor of building is made of reinforced concrete. Blast furnace cement was used for all constructions.—A. E. BEITLICH

The necessary reinforcements for reinforced concrete girder slabs. KURT BERNHARD. *Zement* (Germany), April, 1930, V. 19, No. 16, 17, 19 and 20, p. 379-83, 405-9, 447-52 and 474-8.—Formulas and equations are given for calculation of necessary reinforcements for girder slabs. Steel coefficient can be calculated and amount of steel determined for slabs of any cross section. All stresses are analyzed. Examples show calculations for single slabs and slab ceilings. Special consideration is given to different fields, end and center fields, with varying number of supports.—A. E. BEITLICH

Simplified calculations of flexible slabs. "Die vereinfachte Berechnung biegsamer Platten." MARCUS. 2nd edition. Julius Springer, Berlin, 1929, R. M. 11.00. Reviewed in *Zement* (Germany), April, 1930, V. 19, No. 17, p. 410.—Book gives calculations and describes stresses in all kinds of slabs and ceilings, and examples show that simplified formulas are well suited to the calculation of such work.—A. E. BEITLICH

Moments in continuous slabs and their reinforcements. SCHNEEMANN. *Zement* (Germany), May, 1930, V. 19, No. 21, p. 497-9.—Calculations are given, which show different moments in continuous slabs. Necessary change of standard specifications which increases support moments and reinforcements is suggested.—A. E. BEITLICH

Reinforced concrete ties after system Emperger. *Zement* (Germany), May, 1930, V. 19, No. 21, p. 499-502.—Weight of wooden ties is in general 75 kg., while concrete ties weigh 150 kg. This greater weight is more suitable to stand high vibrations. A special type of concrete ties is described, which consists of three parts. Transportation and placing of these ties is easier than ties consisting of one piece only. Reinforcements provide flexible connections of three parts. Iron parts are protected against rust by coatings or special arrangements. Drawings and photographs show all details, and practical experiences are given.—A. E. BEITLICH

Nomogram for determining the area of longitudinal steel. JORGE QUIJANO. *Ingenieria* (Mexico), Jan., 1930, p. 37.—A simple alignment or nomographic chart, one of a series of such charts for use in reinforced concrete design, solving the relation $A_s = pA_c$. Left hand scale or line Ac, middle scale As, and right scale p. All in metric units. Explanation of use is given on full page chart.—MILES N. CLAIR AND C. G. CLAIR

Nomogram for calculating bond stress. JORGE QUIJANO. *Ingenieria* (Mexico), Jan., 1930, p. 30-31.—A simple alignment or nomographic chart, one of a series of such charts for use in reinforced concrete design, solving the relation $uEo = Vb$. Left hand scale of line $\frac{u}{V}$, middle scale $\frac{As}{b}$ and right scale

diameter of bars. Explanation and examples of scale are given for determining various factors. All in metric units.—MILES N. CLAIR AND C. G. CLAIR

High unit stresses recommended for concrete-cased I-beams. *Eng. New Record*, May 15, 1930, V. 104, No. 20, p. 798-799.—A recommendation for 24,000 lb. unit stress in steel I-beams of floor framing when cased in reinforced concrete and covered with slab of same material is main conclusion of report presented on May 12 to Western Society of Engineers by committee which, in co-operation with Chicago building department, has conducted tests on floor construction of this type. Recommendation applies to I-beams up to 24 in. in depth when wrapped in steel wire fabric or mesh and incased in rectangle of concrete with minimum covering of 2 in. on flanges of I-beam and when used in connection with concrete floor slab not less than 2½ in. thick above top of flange of I-beam. Committee presented following facts based on study of number of tests: (1) Concrete and structural steel act together to form composite beam. (2) Influence of concrete prevails under vibration and impact. (3) Observed stresses in steel and deflections in slab are far below those computed on assumption that entire load is taken by steel alone. (4) Considerable economy in support of live loads may be obtained by utilizing joint action of concrete and steel. (5) Within practical limits composite beam conforms to theory and may be designed by methods analogous to those used in case of reinforced concrete T-beam. (6) Added strength of concrete may be taken advantage of in design by simple method of adopting higher unit working stresses in steel and then, with these stresses, calculating steel alone.—D. E. LARSON

Layout and construction of a new wet-process cement plant in Texas. TERRELL BARTLETT. *Eng. News Record*, May 29, 1930. V. 104, No. 22, p. 889-893.—Structural and manufacturing features of recently completed wet-process cement mill near San Antonio, Texas, for Republic Portland Cement Co., involve the following factors of interest: site selection; easily available quarry material in one place; layout of plant to reduce costs to a minimum; use of an unusual amount of concrete in the structures; concrete craneway frame; foundations on both rock and clay; large grinding capacity; close control, and silo construction for stock house. Because of the nature of the industry use of concrete was considered to be good policy wherever its cost would be equal to or less than steel. Steel is costly in Texas because of freight charges and concrete is cheap on account of labor conditions. The concrete craneway is unusual structural feature. In designing beams at the top it was necessary to consider impact of loaded crane weighing 50 tons and transverse thrust resulting from movement of crane trolley. In designing for wind load, each side frame was made laterally self supporting by use of buttress shaped columns on spread footings resting on rock. Columns of each frame are connected at mid-height by row of double-corbelled concrete struts. Analysis indicated that expansion joint was necessary in 300-ft. frame length to prevent undue temperature stresses in end columns.—D. E. LARSON

Curves for analysis of rectangular continuous beams with axes semi-circular in plan. ST. HESSLER. *Beton u. Eisen* (Germany), April 20, 1930, V. 29, Heft 8, p. 149.—Moment curves are given for torsion and direct bending in uniformly loaded beams, continuous over two, three, and four equal spans, for both fixed and hinged condition of end supports. Reference is made to article by author (cf. *Beton u. Eisen*, Heft 23, 1927) for treatment of theoretical basis of curves.—A. A. BRIELMAIER

Model tests of a flat-slab crossing structure. A. BUHLER. *Beton u. Eisen* (Germany), April 20, 1930, V. 29, Heft 8, p. 154.—This is brief summary of some tests made by Swiss Federal Railways. Model consisted of two spans with an overall length of 40 ft., width of 7½ ft. and height of 10 ft. Intermediate support was column with flared head and end supports were vertical

thin slabs with knee strengthened by bracket similar to part of center column head. Structure was loaded to failure with lead billets, and comprehensive set of observations made on several phases of structural behaviour. Among the conclusions the following points are mentioned: Cracks can not be prevented but may be kept small by proper distribution of steel. For usual steel ratios and concrete of fair quality ultimate load is dependent on yield point of steel. Shrinkage in concrete should be considered as about 0.05 per cent (Swiss Reinforced Concrete Standards call for 0.025 per cent). Use of reinforcement with high elastic limit is definite advance in concrete construction. Steel used in test showed elastic limit of from 57,000 to 77,000 lbs. per sq. in. From construction viewpoint flat-slab model was not very satisfactory. Knees at end supports were subjected to irregular stresses and early cracking. Exact placing of steel was difficult. E value for concrete showed a variation of ± 20 per cent, and decreased with increase in loading. It increased by about 6 per cent between ages of 1 and 3 months. Plane sections do not remain planar in highly stressed regions. Permanent deflections became evident at loads equal to two or three times design loads. Test cubes showed considerable variation in strength. Compressive and flexural tests of prisms showed permanent deformations at low stresses for young concrete. Possibility of permanent deformation as well as strength should be considered in determining time limits for removing forms.—A. A. BRIELMAIER

Successful reinforced concrete base for street railway tracks. I. O. MALL. *Concrete*, April, 1930, V. 36, No. 4, p. 13.—Methods followed in development of reinforced concrete track support in Canal Street, New Orleans, is combination of careful structural design supported by thorough tests under conditions simulating actual service. Roadbed structure consists primarily of reinforced concrete foundation to which rails are anchored by means of resilient rail anchorages. Foundation slab is shaped to form continuous girder beneath each rail and parallel to rail. The reinforcement in each of these girders consists of two $\frac{5}{8}$ in. straight and deformed bars. The transverse reinforcement, extending the full width of slab and passing under each rail, is also of $\frac{5}{8}$ in. round deformed bars. Two continuous longitudinal angles are embedded in concrete beneath and parallel to each rail. Rail anchorage units are fastened to these angles, which are held in parallel alignment by deformed straps welded to angles at points of contact. Tie rods, which extend from rail to rail, are covered with an asphaltic material to prevent their bonding to concrete. The cushion between rail and pavement along its side is accomplished by application of $\frac{1}{4}$ in. coat of asphalt to rail before concrete is placed. Transmission of shock from rail to concrete base is prevented by placing $\frac{1}{4}$ in. thickness of elastite immediately under base of rail, this strip being held in place by special wire clips. Further elimination of impact is obtained by means of a specially designed anchor plate or diaphragm.—C. BACHMANN

Concerning concrete and cement. R. E. *Roads and Rd. Constr.* (England), June 2, 1930, V. 8, No. 90, p. 196.—Referring to various Building Science Abstracts, the author stresses need for taking into account contraction stresses in reinforced concrete structures. Contraction per unit length varies from .0002 at three months to .0005 after six years, when setting stress will be 128 lb. per sq. in. For piles in water bearing soil, permissible load in tons is given by $P = Wh/(S + 1)$ where W is the weight of monkey in tons, h the fall in ft., and S the average penetration in inches for the last five blows. For power rams $P = Wh/(S + 0.1)$. Additional supporting power may be obtained by fitting to head of pile a 5 ft. square slab collar. Regarding cement guns, it is found that allowance for bulking should be made. If 1 to 3 dry sand mix is specified, 6 per cent moisture in pit run sand increases the bulk by 33 per cent and cement ratio will become 1 to $2\frac{1}{4}$ instead of 1 to 3. Furthermore it is found that when mix is applied to hard surfaces, 25 per cent of sand is lost through rebounding due to its velocity. Ratio of placed mix, then be-

comes 1 to 1.68. For coloring concrete, organic pigments should be avoided in favor of inorganic pigments such as ochres, umbers and oxides of iron, and synthetic colors like ultramarine. These, which should be finely ground as cement, do not decrease the strength.—John E. Adams

Architectural Design

Concrete architecture. Willi Lutz. *Zement* (Germany), May, 1930, V. 19, No. 19, p. 445-7.—Concrete in hands of an artist has many advantages over natural building materials. It has highest plasticity and durability and can be placed without joints. Molds can be used many times. No material waste. Selection of aggregates and admixtures allows variations of color and texture according to its purpose. Application of concrete in artistic architecture is illustrated on several reconstructions of big warehouse showing its use for facing of stairways and outside structures.—A. E. Beitlich

Concrete beautiful. Ernest O. Bostrom. *Western Architect*, May, 1930, p. 72-73.—An architect's appreciation of concrete as an architectural medium combining strength with form and color.—Rexford Newcomb

New concrete plastics. Dahl. *Zement* (Germany), June, 1930, V. 19, No. 23, p. 540-1.—Modern building construction uses more and more methods to provide outside of buildings with decorative finish and ornamental plastics from the hands of an artist. A number of photographs show what can be done along this line. Mtehods are described and suggestions for their application are given.—A. E. Beitlich

Construction and shaping of skyscrapers of reinforced concrete. (See Field Construction—Buildings.)

Field Construction

Bridges

Erection of caisson of Poughkeepsie suspension bridge. H. G. Schwegler. *Der Bauingenieur* (Germany), Jan., 1930, V. 11, No. 5, p. 71-2.—Highway bridge over Hudson river is 900 m. long with suspended portion of 450 m. Two main towers rest on 19.8 m. high concrete piers which are placed on concrete caissons. Dimensions of caissons are: 40.8 m. long, 18 m. wide and 31.2 m. high. Difficulties were encountered in placing one caisson. Upper part leaned over during construction under angle of 43 degrees. Erection of caisson which weighs 11,000 tons is described.—A. E. Beitlich

Thin bridge floors concreted in Vermont winter. H. L. Tilton and F. B. Saunders. *Eng. News-Record*, June 19, 1930, V. 104, No. 25, p. 1002-1005.—In winter of 1928 Vermont department of highways was confronted with task of completing large number of 1200 bridges which were destroyed by great flood of 1927. Completion of these bridges was necessary to relieve temporary structures which would be swept away with breaking up of ice. Concrete work consisted mainly of piers, abutments and floor slabs and was made difficult by temperatures as low as —20° F. In placing concrete in piers and abutments a 2 per cent solution of calcium chloride was incorporated in 1:2½:5 mix. Materials were heated by steam, furnished in one case by steam shovel previously used for excavation. Steam jet was used in water tank and steam pipe grid under sand and stone piles. Kerosene flame heater was used in mixer. Heating operations were started 24 hours before concreting. By these methods concrete was discharged from the mixer at temperature of 75° or higher. Freshly placed concrete was protected by canvas placed over top and down sides with outlets for live steam between canvas and forms. A

temperature of 50° was maintained even in severe weather. Heat was removed after 48 hours and as late as ten days after placing, the surface of concrete was still warm when forms were removed. Floor slabs of 1:2:4 concrete 5 in. to 9 in. thick were required by specifications to be inclosed and heated for 14 days, but to expedite opening of bridges and to decrease cost of heating, high-early-strength cement was used. Materials were heated as described above. Floor slabs were protected by canvas inclosure below. Heat was supplied by coke-burning salamanders and was confined entirely to canvas inclosure. About 125 sq. ft. of floor area was allowed per salamander with extra salamanders near the walls on windy days. Use of live steam instead of salamanders was soon abandoned. As soon as the concrete was screeded and broomed, building paper was laid on it. Then a covering of hay was spread on the paper and battened down with boards. By applying the paper directly to the concrete surface heat was retained uniformly in concrete. During first 24 hours temperature beneath paper on top of slab was from 2° to 3° higher than that in heated inclosure underneath slab. Test beams 6 in. x 6 in. x 36 in. were made from concrete in slab. These were laid on the slab and cured under identical conditions. Heating was stopped 24 hours after test beams developed a compressive strength of 2000 lb. per sq. in. This strength developed in 48 to 72 hours. Due to variable moisture content of aggregate, smallest amount of water to produce workable mix was specified. The cost of winter protection varied from 10 to 15 per cent depending on the size and location of the work.—ROBERT W. ABBETT

Subsiding bridge. *Roads Rd. Constr.* (England), June 2, 1930, V. 8, No. 90, p. 198.—Part of foundations to bridge over Salwarpe, Worchestershire were found to be undermined by scour of stream. Divers discovered earth and subsoil had been removed from under one abutment to depth of six feet for length of 16 ft. and 7 ft. back from face. Concrete in bags was inserted by divers, until whole cavity was filled, and adjacent part of river bed also made good with concrete in bags, and block stone.—JOHN E. ADAMS

Welded trusses developed for bridge floor. (See ENGINEERING DESIGN —Bridges.)

BUILDINGS

Construction and shaping of skyscrapers of reinforced concrete. "Konstruktion und Gestaltung grosser Geschossbauten in Eisenbeton." STORTZ. 1930, Konrad Wittwer, Stuttgart (Germany), R. M. 6.00. Reviewed in *Zement* (Germany), May, 1930, V. 19, No. 18, p. 432.—Details of skyscraper construction are discussed and suggestions are given for architectonical effects and their construction. Book is accompanied by many valuable illustrations.—A. E. BEITLICH

Construction of reinforced concrete tower for Navy Monument in Laboe near Kiel (Germany). WILHELM HAEVERNICK. *Der Bauingenieur* (Germany), Feb., 1930, V. 11, No. 7, p. 110-2.—Tower is 68.20 m. high, 13.80 m. wide and 31.50 m. long. Under ground consists of 5 m. thick layer of marl, then loam, 3 m. thick layer of sand and marl again. Structure rests on 1 m. thick cross reinforced concrete foundation. High quality iron portland cement and high alumina cement from Hochofenwerk Luebeck were used for construction of 10-story building. Concrete pump was used for delivery of 8 to 10 cu. meters of concrete per hour.—A. E. BEITLICH

Design and construction of the main building of the German Health Museum in Dresden (Germany). CURT G. SCHINKE. *Der Bauingenieur* (Germany), March, 1930, V. 11, No. 13, p. 229-32.—Main building which is 30 m. high is reinforced skeleton structure. Covered area is 43 m. by 50 m.

Building consists of two halls, one of them being 14.8 m. high, rooms for conventions, rooms for microscopical work, library and several rooms for lectures and exhibitions. Interesting feature is ceiling construction of big hall. Detailed description is given of formwork, concrete transportation, distribution and placing.—A. E. BEITLICH

A reinforced concrete skeleton structure in Breslau (Germany). OTTO VEIT. *Zement* (Germany), May, 1930, V. 19, No. 18, p. 429-30.—Bad underground conditions made foundation on concrete piles necessary on which reinforced concrete structure of 10-story building rests. A detailed description of the construction is given.—A. E. BEITLICH

German museum in Munich. L. HOFFMANN. *Zement* (Germany), May, 1930, V. 19, No. 20, p. 470-2.—Entire building project is divided into three parts. Building which contains collections and expositions was finished several years ago; buildings for library and conventions is under construction now. Latter structure has total length of 400 m. and has several large halls; one of them, which gives room for 2000 people, is 14 m. high. Many difficulties were encountered in placing foundations. It was necessary to make 520 small excavations which were filled with concrete on which steel structures of building rest. Reinforced concrete was used for most all of the superstructure. Ceilings with great spans were placed; roofs are covered with metal.—A. E. BEITLICH

Directions for construction of reinforced rib-ceilings with hollow bricks of pumice concrete. *Zement* (Germany), May, 1930, V. 19, No. 19, p. 455-6.—Great care must be taken for concrete forms. Bricks must be thoroughly wetted before being placed. It is necessary to provide for openings in the forms for pipes in order to prevent damage by later chiseling.—A. E. BEITLICH

The new construction of the building for the newspaper "Neue Mannheimer Zeitung" in Mannheim (Germany). NORKAUER. *Zement* (Germany), May, 1930, V. 19, No. 18, p. 427-9.—Building was erected in reinforced concrete skeleton construction. High early strength cement was used. Frost periods, which exposed structure to —28° C. for a long time, did no damage to building. Details illustrate the construction of foundations for heavy machinery.—A. E. BEITLICH

Concrete bricks for building construction. *Zement* (Germany), April, 1930, V. 19, No. 16, p. 384-5.—New form of concrete brick with porous core and hard outer shell is described. Mixture of one part of cement with ten parts of aggregates (gravel, slag, volcanic ash or pumice) are used for their manufacture. Form of bricks makes construction of any kind of wall possible. An empty air space is formed in interior of wall which provides good heat and sound insulation. Costs of construction can be reduced to about 50-60 per cent of the costs of ordinary brick masonry.—A. E. BEITLICH

New Storage warehouses of the Paris-Austerlitz station. CH. DANTIN. *Genie Civil* (France), Jan. 11, 1930, V. 96, p. 29-33.—To eliminate delays on a job comprising an area of 23,000 sq. m., great care was taken to provide satisfactory forms for all parts of the structure and to insure mechanical handling of materials. Rational choice of concrete proportions and careful supervision during hardening led to successful completion ahead of date set. Proportions were: 600 liters sand, 600 liters gravel and 350 kg. cement. To obtain a check on progress of hardening, samples were taken at the time of placing each member. These specimens were tested in flexure by means of a machine of lever type and removal of forms was postponed until strength values exceeded three times theoretically allowable limit. A table was posted giving safe stress and machine reading desired for each member.—MRS. M. A. CORBIN

Control practice in reinforced concrete construction in erection of warehouse No. VII in Free Port of Stettin (Germany). HERMANN

CANTZ. *Forscherarbeiten auf dem Gebiete des Eisenbetons* (Germany), 1930, V. 34, 71 pp.—Introduction deals with general tasks of construction control of concrete and reinforced concrete jobs. Example shows control work for construction of 5-story warehouse. Preliminary tests were made in laboratory erected in neighborhood of place of construction. Properties of cement and gravel, proportioning of mix and consistency were studied, especially from economical viewpoint. Sand which was used for construction contained up to four per cent loam. In order to find out if this has any unfavorable influence on concrete, tests were carried out with sand to which 12½ per cent of loam were mixed. No decrease in strength could be observed with this admixture. All construction materials were examined, and cement was used only when its 3-day strength was satisfactory. Sand was tested for its gradation, contents of loam and organic materials. Sodium hydroxide solution of 3 per cent was used for latter purpose. Investigations concerning consistency of mortar and concrete showed certain relations between necessary amount of cement and gradation (voids and surface) of aggregates. Method was developed to determine w/c ratio for any aggregates for a given consistency when amount of cement is given. Influence of low temperatures on compressive strength of mortars was studied. Even temperatures from 0° to +8° C. showed retarding effects. Formula was derived for calculation of 28-day strength of concrete for different mixes. Strength of concrete depends on strength of cement, w/c ratio and gradation of aggregates. Special consideration was given to influence of coarse aggregates and relation between cement and concrete strength. Quality of concrete was tested during construction by means of test cubes and beams. Construction control was also applied to mixing, working and finishing of concrete. Special equipment was made for exact measuring of construction materials. Detailed description is given of concrete distribution arrangement. Special care was taken in protecting concrete over night. All parts of building were tested after construction under heavy loads and deformations measured. Entire building occupies area of 47 x 218 m. Suggestions are given for erection of testing laboratories near the job. Thirty-five illustrations and twenty-three tables are given, and several references complete the article.—A. E. BEITLICH

How to save in concrete form work. A. B. MACMILLAN. *Concrete* April, 1930, V. 36, No. 4, p. 27.—Description of forms for girders, beams and floor slabs refers to what is ordinarily termed beam-and-girder construction. In general, following lumber sizes are used: (1) For beam and girder bottoms, 2-in. plank dressed down to 1¾ in. by the width of the beam. (2) For beam and girder sides, 1⅛ in. dressed. (3) For boards of floor form panels, ⅞ in. dressed. (4) For ledgers and ribbands, 1 x 6 in. nominal size, rough or dressed (5) For cleats (or battens) on beam and girder sides, 2 x 4 in. nominal size, rough or dressed. (6) For floor form joists, 2 x 6 in. to 2 x 10 in., depending on load and span. (7) For posts and heads of jack frame, 3 x 4 in. nominal size, rough or dressed. Girder forms are first to be set up. Both girder forms and beam forms are made in three parts, namely bottom plank and two sides. Ends of girder forms are supported by blocking up from uppermost column yoke, or on special framing of 2 x 4's fitted around opening in the column form. Sides of girder are notched out for beam. Girder sides should be cut 2 in. less than length of bottom plank. Girder sides and bottom are assembled separately Boards of sides are nailed to 2 x 4 battens or cleats and are set into place, and bottom plank is likewise set into place. Beam forms are handled similarly. Beam form ends are supported on 2 x 4's nailed to sides of girder forms at level of bottom of beam sides. Sides are 2 in. less in length than the bottom plank and 1 in. shortage at each end is again filled in by beveled 2 x 4's nailed against girder sides with double-headed nails.—C. BACHMANN

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, May, 1930, V. 36, No. 5, p. 35.—Layout for typical interior floor bay shows seven of floor form panels marked F-1, signifying that they are all of same size and shape

and are interchangeable. Six other panels are marked F-2, which means that they are identical in size and shape and interchangeable. A special panel only 10 in. wide, marked F-3, is placed just off center line of floor bay, and is put there as aid to shoring of slab. Typical exterior floor bay layout contains greater variety of floor form panels. With such layouts in his possession, and knowing details, form carpenter foreman will know exactly how many floor panels of each type he will need for each of bays. With posts and girts in place, joists are set in place, but are not nailed or otherwise fastened to girts. Provision is made for substantial "permanent" shores near center of floor bay, by means of special floor form strip or panel. Before any of form work of floor bay is removed these shores are set into place under panel F-3 and fastened with wedges. Rest of form work in bay may then be removed as soon as it has attained sufficient strength to be self-supporting and to support weight of form work on floor above.—C. BACHMANN

DAMS

New Pardee dam a triumph in concrete construction. L. M. JORDAN. *Concrete*, June, 1930, V. 36, No. 6, p. 27.—Beginning of work of constructing Pardee dam across the Mokelumne river near Valley Springs, Calif., consisted of construction of flume to take care of river waters, and work of excavation for foundation was successfully conducted without interruption during winters of 1927 and 1928. Dam is 358 ft. in maximum height, 241 ft. in greatest base width, and has crest length of 1,337 ft. The contract called for completion of dam within 42 months. It was actually completed 1½ years ahead of schedule time. Cement was transported to dam site over 5-mile branch line of railway that was built by the contractor. Concrete aggregate material was secured from tailings of placer mining operations 4 miles down river. After gravel had been excavated by large dragline and transferred on cars to base of washing tower, it was elevated by means of a belt conveyor, and passed down through screens in washing tower. During months of August and September, 1928, the 18,700-ft. direct aerial tramway transferred concrete to dam at rate in excess of 67,000 cu. yds. per month. It had been designed for 440,000 lbs. capacity of gravel, or 147 buckets per hour. During 79-day period it transferred some 232,800 buckets of material from washing plant to bunkers. Four electrically-driven drum mixers were used, inclined into discharge chutes that concentrated materials into control hopper positioned on the hillside below plant, which discharged into a chute terminating at the base of principal concrete tower. Daily moisture determinations of sand were made from which moisture additions were calculated. Slump tests were practiced to arrive at the total water, but judgment was based principally upon the appearance of the concrete.—C. BACHMANN

Construction of weir in Chemnitz (Germany). R. PAPP. *Der Bauingenieur* (Germany), Jan., 1930, V. 11, No. 4, p. 55-8.—River Chemnitz is damned near site of electric power station in order to furnish sufficient water for steam boilers and cooling system. Station produces 84,300 kw. with 24 steam boilers which consume 8,000 to 12,000 cu. meter of water per hour. Old weir was replaced by new modern structure, construction of which is described. —A. E. BEITLICH

Dams of Oued Fodda and Ghrib. M. TRUFFOT. *Annales des Ponts et Chaussees*, (France) Jan.-Feb., 1930, No. 1, p. 41.—Two dams, of which former was begun in April 1926, and latter during second half of 1927, constitute a vast program of irrigation of Chéliff plain in Algeria. Total capital invested is estimated at 250 million francs. The energy produced will amount to about 14 million kilowatt-hours annually. Oued Fodda dam is a gravity dam of concrete with triangular section, 5 m. wide at the crown and 72 m. wide at the toe. It is 200 m. long. The volume of concrete to be placed is approximately

270,000 cu. m. The proportion of cement varies from 150 to 300 kg. per cu. m. of concrete. Expansion joints with copper plates are spaced every 18.6 m. The rock is transported from the quarry a distance of about 400 m. It is screened and stored in silos with automatic discharge, which permit mechanical proportioning of concrete. Proportions are selected with view to avoiding segregation in chuting concrete. Density of concrete varies from 2.4 to 2.6 and 2.7. Kieselguhr in quantities equal to 3 per cent by weight of cement is mixed with latter to increase strength of cement. No consistency tests are used—the only cue is the penetration of a man's foot (shod) up to the ankle. At the time of writing 350 to 400 cu. m. were being placed per day; it was expected to raise this production to 600 to 700 cu. m. per day.—MRS. M. A. CORBIN

Construction of the dam of San Joaquin. ADOLFO ORIVE DE ALBA. *Ingenieria* (Mexico), Jan., 1930, p. 22-25.—Methods employed in construction of an earth dam with reinforced concrete core wall are described. The upstream embankment was constructed with layers 20 centimeters thick of three parts of clayey earth and one part each of sand and gravel. Layers were given slope of 2½ per cent toward center of dam for about 10 meters and then levelled with idea that any tendency toward slipping would be in direction of center rather than toward outside. Downstream embankments differ in that proportions of sand and gravel, in each layer, increase from center to facing, which contains no clay. Clayey earth was excavated by power shovels about 500 meters upstream of dam, and conveyed by dump trucks to place. Sand and gravel were excavated by hand 50 meters upstream, and conveyed by wheelbarrows and baskets to place. Mixing and compacting of materials were done by tractor plows and rollers, with the mass wet but not fluid or muddy. All of the placing of earth fill was done under the supervision of inspector responsible for obtaining proper mixture and compactness. Average cost of embankment was $1.60 per cubic meter. Stone drain at foot of downstream face of dam was constructed of tailings and heavy gravel obtained from clearing bed of river at dam site. This material was loaded into special gondola cars by hand and dumped in place. The average cost in place of such stone transported 80 meters was $0.62 per cubic meter. Careful examination was made of the clay strata in which core or cut-off wall was to start. It was discovered that strata on which dam was originally founded in 1927 was underlaid by loose layer of water-bearing sand and gravel, which might have endangered the dam. Materials for concrete were thoroughly investigated before start to determine proper proportions. Abrams' curve for water-cement strength relation was taken as guide for the following proportions adopted as standard after experiment: 26 litres of water, 50 kilograms of cement, 63 liters of sand, 114 liters of gravel, and an eight centimeter slump. Two plant set-ups were made for pouring core wall. The first set-up used two one bag mixers, to which sand and gravel were fed by gravity from overhead volume batchers which were filled by wheelbarrows. Automatic batch meter was used, and a three minute mix required. Concrete was carried to dam in special gondola cars and then dumped into inclined chute, which in turn fed vertical tube. The second set-up became necessary after about 1000 cubic meters of total 2600 cubic meters had been placed, when concrete reached height of gondola cars. New set-up used only one mixer of two-bag capacity; and concrete was transported to dam by means of cable-way above crown of dam. This cable-way, not only supported gondola car, but also, chute into which it dumped concrete. Concrete was poured continuously day and night, and amounted to approximately 30 cubic meters per eight hours. Difficulty was encountered in getting form work properly and efficiently set up, despite careful preliminary design, because of

inexperience of available carpenters. A chart of the organization, which functioned well, is given.—MILES N. CLAIR AND C. G. CLAIR

Plant producing ready mixed concrete and cement products. (See SHOP MANUFACTURE.)

Concrete molds. (See SHOP MANUFACTURE.)

MISCELLANEOUS

New method of conveying plastic concrete. GEORG GARBOTZ. *Tonind. Ztg.* (Germany), March 17, 1930, V. 54, No. 32, p. 372.—Conveying concrete from mixer to forms, by pumping through pipes, has often been unsuccessfully attempted. This "Pumpkret" process, as developed by Giese and Hell in Kiel, has been found very satisfactory on several jobs. Importance attached to this process is evidenced by formation, under leadership of Torkretgesellschaft, of an international group representing American, English and French interests, to perfect the process. Mechanical means now in use for conveying concrete, carts, cableways, chutes, and bucket belts are unsatisfactory either because of size or because of original and maintenance cost of plant. Chute method has also disadvantage of lowering quality of concrete due to extra water required. The Ransome Company with its "Concrete-Placer" conveyed plastic concrete through pipes by means of compressed air. Pneumatic method of Torkretgesellschaft is based on entirely different principle. Cement and dry aggregate, instead of wet concrete, are blown through pipes to point of pouring. At this place a water chamber provides necessary water. Bongardt process is similar to this, except that cement and aggregate are slightly dampened and are forwarded under lower pressure. Torkret process still has some disadvantages: comparatively high energy consumption and tendency to produce concrete of variable quality. In Giese "Pumpkret" method concrete is mixed in usual manner and flows into feed chamber, at the bottom of which is stirring arrangement that prevents segregation and passes concrete to vertical pump cylinder. Pump delivers it to air chamber from which it is forced through pipes with regular impulses. Pipes are of 2 mm. (0.078 in.) material and have no sharp bends. Vertical portions of line are attached to wooden frame or post and horizontal portions are placed on forms. Operation of equipment is simple. Motor, pump, feed-chamber, agitator, air chamber are mounted on four-wheeled frame. Erection of piping requires only few hours. One man is needed at pump and feed-chamber and one or two men at discharge end of pipe line. Aggregate used is not larger than 20 to 30 mm. (0.79 to 1.18 in.). Consistency of poured concrete is such that it barely flows around reinforcement. Concrete is of good quality and shows no evidence of segregation. Energy consumption is low compared to other processes: 12 H. P. for a 10 cubic meter (13.1 cu. yd.) per hour production delivered vertically 15 meters (49 ft.) and horizontally 15 meters. Against advantages of easy erection, small space requirements and simple operation there exist at this time disadvantages of limited size of aggregate and limited production, difficulties that should be overcome through further study. "Pumpkret" process represents definite advance in reinforced-concrete construction since it presents a conveying method more satisfactory from both economic and scientific viewpoint than any method heretofore employed.—A. A. BRIELMAIER

Instructions for concrete construction. Belgian Association for Standardization, 2nd edition, Aug., 1929. Reviewed in *Annales des Ponts et Chaussees*, (France), Jan.-Feb., 1930, No. 1, p. 65.—Belgian Association has attempted to work out a type of specification applicable on smaller jobs. Values are given for loading, shrinkage, variations in temperature. In discussing stresses, two methods are suggested. Former, in which no tests are made, makes use of four basic concrete mixes, stipulating grading of gravel, natural sand and varying proportions of portland cement, the Abrams cone test being used as indication of consistency. The allowable stresses are given for each of the 12 mixes

thus obtained. In second method, tests are made on 20 cm. compression cubes made under job conditions. The aggregate should be hard and clean; the choice of consistency may vary within certain limits of slump test. Ratio by weight of cement to water should exceed 1.30. Concrete should contain at least 250 kg. cement per cu. m. measured in place. It should be sufficiently compact and plastic to insure proper coating of reinforcement and avoid rust. Ratio by weight of inert particles smaller than 5 mm. to those greater than that size should be within 0.30 to 0.80. Allowable stresses will be equal to one third of the compressive strength of the cubes made in field (average of three tests) and tested at ages of 28, 14 and 7 days. Maximum allowable stresses given for four different cements vary from 50 to 90 kg. per sq. cm. Three test cubes should be made each day for first 15 days; whereupon three cubes are made each week. In no case should stresses allowable in reinforced concrete exceed those of plain concrete by more than 50 per cent. Allowable shearing stress in concrete, computed disregarding stirrups and inclined bars, should not exceed one fifth of the allowable compressive stress. Allowable bond stress should be: one tenth of the allowable compressive stress, when the ends of bars are not hooked; one fifth when hooked ends are provided. In former case stress is not to exceed 6 kg. per sq. cm.—in latter 12 kg. per sq. cm. Tensile and compressive stress of reinforcement may reach 1200 kg. per sq. cm. All parts of construction exposed to repeated load should have allowable stresses reduced accordingly, this reduction not to exceed 25 per cent. Cement used is to be one of three brands of portland cement, slow-setting, or special slag cement. If latter is used, field test method must be followed. Working of concrete shall continue until further mixing produces no change in consistency. Unless otherwise specified, tests of finished structure will be made at expense of the owner.—MRS. M. A. CORBIN

The drillhammer. "Der Bohrhammer." (New journal) Flottmann, A. G., Herne i. W. (Germany). Reviewed in *Zement* (Germany), May, 1930, V. 19, No. 20, p. 480.—Journal covers field of use and application of drilling in bridge and building construction, mines and quarries. Descriptions are given of jobs, and new inventions and improvements of equipment are discussed.—A. E. BEITLICH

Concrete as building material in American foundation jobs. H. GRIESEL. *Zement* (Germany), April, 1930, V. 19, No. 17 and 19, p. 400-2 and 452-5.—Description of methods of construction, which are generally used in America, with pre-cast concrete piles and such piles which are molded-in-place. Advantages of Raymond piles are characterized. Foundations for bridges, dams and large buildings are illustrated.—A. E. BEITLICH

Reinforced concrete for construction of coke plants. *Zement* (Germany), April, 1930, V. 19, No. 17, p. 409-10.—On account of its fire resistance, durability and economy reinforced concrete was used for a great number of modern coke plants. Construction of one is described in details. Storage bins and foundations of ovens as well as all buildings were built of reinforced concrete.—A. E. BEITLICH

Protection of metallic and non-metallic structural materials against corrosion. B. SCHEIFELE. *Der Bauingenieur* (Germany), Feb., 1930, V. 11, No. 7, p. 105-8.—Author illustrates different causes of corrosion of metals, concrete, reinforced concrete, artificial stones and wood and gives directions to prevent such action by successful protective methods.—A. E. BEITLICH

Applications of concrete construction with sliding forms. KNEES. *Zement* (Germany), June, 1930, V. 19, No. 24, p. 561-5.—Description is given of modern methods to save time and to reduce costs of forms and formwork. Special consideration is given to construction with sliding forms, which allows continuous use of one form for construction of bins, stacks, storage houses, tanks and similar structures. Different modern American and German methods are illustrated. Concrete is poured continuously without joints. Control of

homogeneity is very easy. Forms are raised in steps of approximately ¼ in. Average rate of progress is about 7 to 8 ft. per day but can be increased when high quality portland cements are used. Concrete leaves forms after approximately ten hours; after this time it is sufficiently hardened. Weight of forms, platform and laborers rests on foundations. No finishing of concrete surface is necessary since no form marks remain. Several photographs accompany article.—A. E. BEITLICH

Precast concrete caissons for piers and docks floated into place. P. W. LEISNER. *Concrete*, May, 1930, V. 36, No. 5, p. 30.—New patented method in harbor construction, successfully used in building new completed first portion of Polish harbor at Gdynia should be of interest to American builders. About four and a half linear miles of reinforced concrete floating caissons, rectangular in plan, were used in construction. Caissons were towed to their positions in harbor, sunk to rest on prepared sand and gravel foundation and given stability by sand fill. Largest caissons were 105 ft. long, 23 ft. wide and 35 ft. deep, divided into nine cells by vertical cross walls. Width of bottom, or base, was 30 ft. Weight was about 1200 long tons. Most caissons were made standard length of 60 ft., divided into five cells by vertical cross walls. Instead of following the usual practice of building these caissons in dry dock or on launching ways, contractors built them upon surface of land which was to be dredged for harbor basins. Caissons were cast in lying postion on ground, in long rows. As dredging proceeded, finished caissons gradually righted themselves on advancing slope until they floated. Dredging was done with suction dredges.—C. BACHMANN

Selling ready-mixed concrete in Erie. *Concrete*, June, 1930, V. 36, No. 6, p. 13.—Central mixing plant of the Charles H. Fry Construction Co. is well located with respect to receipt of materials and delivery of mixed concrete. A railway spur siding serves this plant exclusively. Washed sand and gravel are bought from pit about 12 miles away. Cement is received by rail, in sacks, and transported by portable conveyors to almost any point in the building. Gravel and sand are unloaded from cars to stock piles and from stock piles to bins. Plant is enclosed and roofed over, except sand and gravel bins at top. These have combined capacity of 18 cu. yd. Mixer, of 1 yd. capacity is charged by gravity from sand and gravel bins above, proportions being made by measure. Normal capacity of mixing plant may be considered 150 cu. yd. per 10 hr. day, but daily production of 175 cu. yd. has been attained. Two transit mixers of 1½ cu. yd. capacity, are supplemented by hired trucks when demand is heavy. Care is taken that water-cement ratio is held down to values consistent with various proportions. Grades of concrete supplied include 1:3:6 mix for footings and foundations in small buildings, 1:3:4 for sidewalks and floors, 1:2:4 for heavy foundation work and structural concrete work in general, and 1:1½:3 for heavily reinforced concrete work. Concrete has been delivered from plant for distances up to 16 miles, requiring about 50 minutes in time. Concrete delivered in city seldom requires haul of more than 3 miles, or 20 minutes. Base price for concrete (at plant) ranges from $7.25 per cu. yd. for the leanest mix, to $9.25 for the richest. The price for topping is from $12.00 to $14.00 per cu. yd.—C. BACHMANN

Concrete benches a necessity in greenhouses. WALTER B. BALCH. *Concrete*, May, 1930, V. 36, No. 5, p. 49.—Greenhouses use two types of soil containers. One known as a bed, is a pit in ground planked up to prevent caving of sides and to hold prepared soil in which plants are grown. Other is known as raised bench, or bench. Concrete beds built at the Kansas Agricultural Experiment Station were made 18 in. high above the ground, 3 in. at the base tapering to 2 in. at the top and extended 6 in. into the ground. Mix was 1:4 river run sand being used. Forms had to be built, but old bed walls and scraps served. For the raised beds, 1¼-in. pipe, set into block of concrete 8 in. square and 4 in. thick, was used for legs. Bottom of bench was made 3 in.

thick, and sides 3 in. thick at bottom, tapering at the top. To carry the load until the concrete had set, ¾-in. angle irons were placed across benches from leg to leg.—C. BACHMANN

Produces and delivers 3,700 cu. yd. of central-mixed concrete a day. *Pit and Quarry*, May 7, 1930, V. 20, No. 3, p. 51-54.—Description of two of Warner Company's three Philadelphia concrete plants—Berks St. yard and Christian St. yard. These receive aggregates by barges, the cement by rail in bulk. Plants are thoroughly up-to-date. Capacity of the Berks St. plant is 1,750 cu. yd. per day, that of Christian St. plant 1,000 cu. yd. per day. Eleven illustrations.—A. J. HOSKIN

West coast firm produces large volume of transit-mixed concrete. R. E. TREMOUREUX. *Pit and Quarry*, May 21, 1930, V. 20, No. 4, p. 57-58.—Describes practices of Golden Gate-Atlas Materials Co., San Francisco, Cal. Aggregates, cement, and water are measured and weighed at central plant, in batches of 4-cu. yd. each. Each batch is discharged into truck mixer which is air-tight when its doors are closed and which revolves enroute to its destination. Average mixing time is 15 min. Each of four cement tanks in plant holds 325 bbl. of cement which is handled from bulk-cement cars by pneumatic pumping. Three illustrations.—A. J. HOSKIN

ROADS AND PAVEMENTS

Concrete streets in Sicily. A. C. *L'Industria Italiana Del Cemento* (Italy), Dec. 1929, p. 8.—Article gives a description of a system of concrete street paving introduced by S. Ghilardi & Co., in which practice similar to that used in the United States is employed. Previously streets were poured in three layers, first of rich mix, second of lean mix of stone mixed with cement mortar, and third of average mix over which was plastered coat of special cement mortar. New procedure has been found to be decidedly better and consists of pouring total thickness as one layer and of one mix, applying surface mortar also contemporaneously or next morning using special quick hardening cement. —C. G. CLAIR AND M. N. CLAIR

Merchants of New Orleans beautify Canal Street. *Concrete*, May, 1930, V. 36, No. 5, p. 47.—In the improvement of Canal St. in New Orleans all old walks were removed, regardless of their condition and new walks 5 in. thick were laid in their place. Top portion was divided into squares, by brass strips, and red border 23 in. wide was made along curb and property line. Coarse aggregate for uncolored portion of top was one part Crown Point spar and one part white marble. For red border aggregate was pink Georgia marble. The red border was obtained by mixing red iron oxide into concrete.—C. BACHMANN

Various municipal works in Bromley. H. CLIFFE. *Contr. J.* (England), June 18, 1930, V. 102, No. 2662, p. 1975.—Concrete carriage ways were constructed on continuous system, a straight joint being left at end of each day's work; expansion joints were provided 150 ft. to 150 yd. apart. It was found difficult to provide satisfactory channel when tamping from the curb so 4 in. thickness of concrete 3 in. wide was laid alongside the curb floated into proper channel levels, a 4 in. by 3-in. timber being placed on such floating and carriageway tamped therefrom. When timbers are afterwards withdrawn, space is filled with fine tarmacadam, the result proving very satisfactory. In carriageways constructed in two widths, 3 ft. long rods were placed at 12 in. centres along joints, and allowed to project 18 in. from first half width laid. Only minimum water for hydration was permitted and concrete was cured with moist sand.—JOHN E. ADAMS

Overcoming paving difficulties in West Virginia. *Concrete*, May, 1930, V. 36, No. 5, p. 13.—District No. 5, West Virginia highway department comprises ten counties lying along the crest of the Allegheny highlands and encompassing the Potomac region. Because of mountainous country and un-

certain subgrades encountered, the district constructs its subgrades at least one year ahead of paving. Slides and movement in roadbed are every-day occurrences. Any pavement laid on new subgrade is almost sure to settle or move out of alignment. Once the subgrade is built, traffic is turned over the road and kept on the new location during the settling period. Grading usually entails moving from 15,000 to 50,000 cu. yds. of material. Much of work is in rock formation, unstable in many cases and prone to slide in others. Curves are frequent, banked as much as 36 in., and are made with as long radius as possible. Embankments are built around curves and along sides of cuts or along drop side of side-wall roads, taking form of heavy-base bank with a curve approaching a parabola. District Number 4, comprising twelve counties, lies to west of District 5, and while its eastern part is mountainous it slopes downward toward the Ohio valley in long rolling ridges. Extensive mining has undermined territory with honeycomb of galleries which cause settling and changing contours of surface land. Peculiar land slides have also occurred, resulting in an unlooked-for elasticity of concrete pavement. This elasticity has led to adoption of somewhat unusual practices in reinforcing slabs. Engineers have come to know just where probable movements will take place, and when another location is impossible extra reinforcing is incorporated. Steel is saved in stretches where it is not urgently needed and accumulated excess is put into slab where movement is likely to occur. Location has been turned to as only means of preventing great expense involved in efforts to overcome force of slides and settlement. Cutting subgrade back into rock or locating right-of-way along ridges or in valley bottoms is engineer's only assurance of permanency.—C. Bachmann

Water Works

New water pipes of public water works of city of Munich (Germany). Lutz Pistor. *Zement* (Germany), May, 1930, V. 19, No. 22, p. 516-7.—Drinking water supply for city of Munich is furnished by wells which produce water of high purity. Two pipes of approximately 50 km. length deliver water to water works. Construction of third pipe was necessary to cover requirements of growing population. Greatest part of this work was done under ground. Cross-section of tunnel was 5.5 $m.^2$ Conditions of ground and surface were very varying. Wall of pipe was 35 cm. thick. Total of 140,000 cu. meters of ground were removed, 60,000 cu. meters of concrete were placed and 120,000 m^2 inner surface were provided with very smooth finish. Aggregates, which were found near the job, were washed free from loam. Wells were drilled to obtain sufficient water for this purpose. Concrete mixer and distributing arrangements were located under ground. Conveyer belts and electric driven cars delivered material to laborers who placed concrete behind forms. Work was continued during winter. Aggregates were heated by means of steam pipes in bins.—A. E. Beitlich

Shop Manufacture

Detroit concrete block plant on large production basis. M. B. Williston. *Concrete*, May, 1930, V. 36, No. 5, p. 17.—Concrete products manufacturing plant of R. E. Hamilton's Sons, Inc., in Detroit, is arranged and equipped for manufacture of all sizes and shapes of solid and hollow concrete units on a large production basis. The company manufactures cinder concrete units in complete line of more than 50 different sizes and styles, as well as art stone, chimney blocks, pier caps, sills, coping tile and related specities. Production capacity is 18,000 units per day of gravel concrete block and 7,000 daily in cinder unit department. Block machines in main building consist of two automatic standard strippers each producing 6 units, and one master tamper producing 4 units per minute. In cinder unit plant there is one automatic super-stripper producing 8 units per minute. Two block racks are set

side by side on a small revolving platform, for removing newly made block. Man removing block from machine fills one rack, when platform is immediately revolved 180° This rotation brings the empty rack next to block-making machine and loaded rack on other side. Louden overhead monorail system is used in transporting newly manufactured units from machine to curing chamber and later from curing chamber to all parts of open-air storage yard. Each trolley has 8 small wheels which run on the lower flange of a continuous I-beam. Entire block manufacturing operations are handled by plant superintendent and 20 men.—C. BACHMANN

Twenty-five different units with one power stripper. *Concrete*, April, 1930, V. 36, No. 4, p. 19.—Two outstanding features about plant of the Crown Sidewalk and Block Co., Minneapolis, Minn., are the kitchen-clean appearance of stock yard and remarkable range of style and sizes of masonry units produced. Twenty-five standard units are made on one Anchor power stripper machine, equipped with quick-change attachments for production of all standard and special sizes, styles and faces required. There are 25 different piles of blocks in yard at all times, displaying as many different kinds or sizes of units.—C. BACHMANN

Concrete molds. *Rev. materiaux construction trav. publics* (France), Feb., 1930, No. 245; p. 69-70.—Molds made of concrete composed of fine sand and cement only are used for manufacture of objects of unusual shapes, to obtain very smooth surfaces and clear cut edges. At moment of making mold, concrete must be very liquid to penetrate into all the interstices. There are several methods of making these molds.—A. SIZE

Plant producing ready-mixed concrete and cement products. *Pit and Quarry*, June 18, 1930, V. 20, No. 6, p. 28-29, 60.W. G. Traver Supply Co. operates two adjacent plants in Decatur, Ill., one producing ready-mixed concrete, the other manufacturing concrete blocks and other cement products. All aggregates—sand, gravel and crushed stone—are received by rail. Mixing plant capacity is 150 cu. yd. per 9-hr. day. Aggregates, as received, are dumped into concrete hopper whence they are taken by a chain-bucket elevator, through divided hopper into two circular wooden tanks, each of 4-carloads capacity, one being used for sand, one for gravel or stone. Each bin has two gates. One gate discharges to trucks in concrete tunnel, the other to volume batcher where cement and Celite are added from sacks. Batcher discharges into 28-cu. ft. mixer, and charge is mixed from 3 to 5 min. Water is measured from a tank having calibrated glass gauge. Cement is bought in sacks and carried from cars to the mixing floor by a slat elevator. Concrete is delivered in trucks with dump bodies. Block plant is new, and is of concrete, concrete-block and structural steel construction. Elevator which supplies the mixing plant supplies this block plant by discharging through a separate chute to inclined belt conveyor which runs to top of building. Materials are discharged into two large wooden tanks, each of which is divided into two sections, thus providing separate storage of sand, gravel, crushed stone, and haydite. Slat elevator previously mentioned feeds a belt conveyor which carries cement and Celite to block plant. Bin compartments discharge to 14-cu. ft. batch hoppers. Hoppers discharge to 32-cu. ft. steel car which serves 21-cu. ft. mixer. Prepared mixture discharges into stripper block machine which has made more than 3,200 blocks in 9½-hr. day. Blocks are loaded onto steel racks and hauled on lift truck into curing kilns where they remain 36 hr. at temperature of 76°. Air is forced through kilns by compressor. Heated water supplies heat and humidity by discharge through valves into fog-like spray. Hydraulic jack is used for testing concrete cylinders and block. Six illustrations.—A. J. HOSKIN

Materials

Admixtures

Influence of admixtures of loam in sand upon mechanical properties of portland cement mortars. M. G. Jeltschaninoff. *Zement* (Germany), June, 1930, V. 19, No. 24-5, p. 556-60, 580-5.—Experiments concerning influence of loam in sand were subject of comprehensive investigation. Following conclusions were drawn: Loam admixtures in sand are in general unfavorable. Sand may be used containing not more than 2 per cent pure loam. Organic materials in loam are of great importance. Tests must be extended over long period, at least several months. Loam should not be dried before tests. Both tensile and compressive strength must be examined. Question of influence of loam is not entirely solved and must be further studied.—A. E. Beitlich

Rhenish trass from the neighborhood of Eifel. K. Biehl. *Ciment* (France), July, 1930, V. 35, No. 7, p. 266-8.—Trass was secured by Romans near Eifel and used in concrete water conduit from that place to Cologne. Parts of the conduit still remain.—F. O. Anderegg

Aggregates

Application of Trilinear charts to aggregate grading control. Theodore E. Sheafer. *Concrete*, April, 1930, V. 36, No. 4, p. 33.—The method of plotting a trilinear chart is explained by example, using four different gradings of sand, indicated by a, b, c and d, the sieve analyses for each being given. The chart is labeled to correspond with the three sieve sizes, No. 14, No. 28 and No. 100. The application of trilinear charts to aggregate grading control is also given in form of a typical example. By carrying in stock three sands of different grading, namely, a coarse sand (C), a sand of medium fineness (M), and a very fine sand (F), the grading of the combined sand (S) can be controlled by selecting the proper proportions of the three sands according to the methods described.—C. Bachmann

Volcanic slag and similar mineral rocks in the region of the Lake Laach (Germany). Otto Gassner. *Zement* (Germany), July, 1930, V. 19, No. 28, 29, p. 658-60, 682-7.—Volcaneous rocks and slags of this region, such as basalt lava, volcaneous slag, tufaceous rocks, pumice and trass are of importance in concrete practice. These materials originate from eruptions of former volcanoes. Their formation and physical properties are described and strength specifications discussed. Special consideration is given to volcaneous slag. Mines are driven into deposits until material collapses. Dump cars transport broken masses to screen which separates them into sizes of 30, 40 and 50 mm. Coarses portion goes through crusher. Conveyer belt delivers material to storage bins. Chemical composition shows great uniformity and no constituents are present, which have unfavorable influence on cement or hardening process. Slag has very good hydraulic properties and shows great resistance to frost. Experiments indicated that concrete with this slag as aggregate also showed great resistance and less decrease in strength when exposed to fires, than concrete which was made with gravel. High heat-insulating properties make it a valuable material for building construction. Data are given for different concrete mixtures and strength properties are compared. A number of further possibilities for use of material are cited. —A. E. Beitlich

Control and improvement of the grading of sand, gravel and pebbles used as aggregate in concrete. Adolf Janousek. *Le Ciment* (France),

June, 1930, No. 6, p. 245.—Author gives report of extensive tests to determine approximate volume and surface area of aggregates. Sieves selected were similar to standard American sieves. The pebbles were taken from bed of the Vetava near Prague. Ellipsoids of three axes, slightly angular, were substituted for round and elongated pebbles. It was found that 138 pebbles of 15 to 30 mm. were equivalent in surface area and volume to 218 imaginary spherical pebbles of 15 mm. diameter. As concrete strength depends not only on dimensions of aggregate, but on surface area as well, this is of great importance, particularly in connection with tensile strength of concrete. Parallel tests were made grading the aggregate in accordance with Fuller's curve, Abrams' fineness modulus and using only fine sand and coarse pebbles in the third series. The most instructive mix was a tamped concrete of "moist" consistency with 3.84 liters water, 60 kg. sand and dry pebbles and 9.6 kg. high test portland cement. While the mixes contained equal quantities of mixing water and cement and had the same fineness modulus, their strengths varied greatly. The author urges that sieve analysis be made a part of regular field tests of concrete. To simplify procedure he recommends a set of five sieves of 1, 3, 7, 15 and 30 mm. for the coarser materials and 1, 2, 5, 10 and 20 mm. for the finer materials. The curves obtained resemble the Fuller curve. The allowable discrepancies would be ±5 per cent of the total weight of the material analyzed.—M. A. CORBIN

The economic use of separate aggregates and the automatic regulation of the water addition to obtain a uniform concrete. OSKAR SPETZLER. *Die Bautechnik* (Germany), Feb., 1930, V. 8, No. 9, p. 130-2.—Advantages of separate addition of gravel and sand for manufacture of dense concrete are described. Quality of concrete depends on (1) quality and amount of cement, (2) quality and gradation of aggregates (3) amount of water and (4) good working and finishing of concrete. Amount of water must be changed with varying moisture contents of aggregates. This change can be prevented when sand is saturated with water before entering mixer. Mixing equipment for accurate measuring of materials and production of uniform concrete is described.—A. E. BEITLICH

Effect of soft particles of coarse aggregate on strength and durability of concrete. W. J. EMMONS. *National Sand Gravel Bull.*, June and July, 1930. V. 11, No. 6, p. 9-16, No. 7, p. 9-14.—Report was first presented before the Sixteenth Annual Conference on Highway Engineering, Ann Arbor, Michigan, and published in Proceedings of that conference. It covers results of an investigation carried out by State Highway Laboratory of Michigan to determine effects of certain soft and non-durable particles, occurring locally in gravels, on strength and durability of concrete. Soft particles studied in this investigation included three classifications of sandstone (hard absorbent, intermediate or borderline, and soft) shale, ochre, and a curious light weight, porous, fossiliferous, dolomitic material, colloquially known as floaters. Gravel used as the basis of comparison was hand picked practically free of soft particles. Concrete specimens were made using standard gravel to which was added 0, 2, 3, 5, 10 and 15 per cent of each of the types of soft particles being investigated. Three groups of specimens were made for each percentage and type of soft particle,—a set of specimens consisting of two beams and three cylinders. First group was tested after normal moist air curing for 28 days; second after 25 alternations of freezing and thawing carried out subsequent to 28 days of normal moist air curing; third group has not yet been tested, being held for long-time tests. Author concludes that "after reviewing data here presented there can be little doubt that presence in concrete aggregates of materials studied is injurious. The normal strength of concrete is reduced and liability of deterioration under freezing and thawing conditions is greatly increased. Much of effect of these materials upon

both strength and durability is dependent upon location of particles in structure, and neither in the laboratory nor in the field can this factor be controlled. A far more extensive investigation, involving concretes of different proportions and exposure under many conditions would be necessary to determine, within reasonable range of accuracy, quantitative measures of these effects under service conditions. That such an extended study would be worth while, is doubtful, for the several general types of the undesirable materials are themselves of rather indefinite and variable classification, and not susceptible to accurate definition. In areas where such soft and structurally unsound materials occur, however, this investigation does unmistakably point to the desirability of reducing. to economic minimum, their percentages in coarse aggregate."—STANTON WALKER

CEMENT

Revised procedure for the determination of uncombined lime in portland cement. WILLIAM LERCH and R. H. BOGUE. *Ind. Eng. Chem. Analytical Edition.* July 15, 1930, Vol. 2, No. 3, p. 296-8.—Revised procedure for determination of uncombined lime in portland cements by ammonium acetate titration method is presented. Various factors which bear on precision of test are discussed. A curve is given which shows that percentage average deviation from mean varies with mean value for uncombined lime. The probable errors have been determined and a curve is given which shows probable error of any group of determinations from mean, at any mean uncombined lime content up to 7 per cent.—R. N. YOUNG

Mortars and cements in ancient history. F. GOLDSCHMIDT. *Zement* (Germany), June, 1930, V. 19, No. 23, p. 543-4.—Asphalt was frequently used as building material in countries where it was found; it was good binder and weather resistant. Gypsum and lime were most common mortars. It cannot be determined whether additions of sand were made or if sand, which was found in the analyses was incidentally in raw materials. In dry countries a favorite mortar was clay or loam, mixed with straw, but only suitable for smaller buildings.—A. E. BEITLICH

Rotary kiln calculation. A. B. HELBIG. *Zement* (Germany), June, 1930, V. 19, No. 24, p. 568-9.—Continuation of polemic between author and Hans Bussmeyer on the subject of comparison of heat economy of "Lepol"-kiln and ordinary rotary kiln with waste heat recovery. (c. f. A. B. Helbig, *Zement* (Germany), May, 1930, V. 19, No. 19, p. 440-4 and H. Bussmeyer, *Zement* (Germany), May, 1930, V. 19, No. 22, p. 522-4.)—A. E. BEITLICH

Rotary kiln calculation. HANS BUSSMEYER. *Zement* (Germany), July, 1930, V. 19, No. 29, p. 680.—Last article concerning polemic between author and A. B. Helbig about superiority of "Lepol" kiln over rotary kiln with waste heat recovery. (cf. A. B. Helbig, *Zement* (Germany), May 1930, V. 19, No. 19, p. 440-4 and June 1930, V. 19, No. 24, p. 568-9; Hans Bussmeyer, *Zement*, May 1930, V. 19, No. 22, p. 522-4.)—A. E. BEITLICH

Control of reactions in rotary kiln. *Zement* (Germany), June, 1930, V. 19, No. 23, p. 536.—Description of sampling device designed by W. H. Lacey and Hubert Woods, which enables sampling of clinker at different parts of rotary kiln during burning process. Samples from front end of kiln were taken by means of long iron pipe with attached sample container. Samples from middle and end of kiln were taken through holes which were drilled through shell and lining. Holes were provided with special caps. Arrangement worked very satisfactory. Determinations of free lime and ignition loss showed rate of combination and enabled burning control. (c. f. *Rock Products*, Jan. 1930, No. 2.)—A. E. BEITLICH

About formation of dusting clinker. J. MUNDORF. *Zement* (Germany), June, 1930, V. 19, No. 26, p. 608-9.—A mixture of 40 per cent blast

furnace slag and 60 per cent limestone was burned in a shaft kiln. Coal was mixed with raw mixture. Produced clinker was very satisfactory until recently when product showed very great amount of dusting clinker. Reason for this behavior was an increasing amount of metallic iron up to 8 per cent in slag which lowered fusion point and prevented air from passing through clinker. Uncomplete combination of raw materials resulted. Burning process was under reducing conditions, an amount of 8 per cent of CO was found in the stack gases. Ferric oxide was reduced to FeO and metallic iron. Since Fe and FeO do not combine with CaO an excess of lime was present which fact made conditions worse. After replacing high iron containing slag by good slag, product was of satisfactory quality.—A. E. BEITLICH

Methods of investigating the thermochemistry of cement. W. A. ROTH, *Zement* (Germany), July, 1930, V. 19, No. 27, p. 628-30.—Reactions under investigation are formation of clinker from raw materials and setting and hardening process of cement. Direct calorimetric measurements are impossible to obtain. Heat tones of these reactions can be studied indirectly by determining heat of solution. Best solvent is mixture of 100 parts of hydrochloric acid (20 per cent) and 15 parts of hydrofluoric acid (20 per cent) which dissolves all constituents. Metal container with inner gold lining can be used as calorimeter. Solution can be accelerated by (1) very fine pulverization in order to expose great surface to action of acid, (2) great agitation in calorimeter, (3) higher concentration of the solvent and (4) starting of reaction at higher temperature than room temperature. Experiments were made at 50 and 100° C. Second way to attack this problem is to study heat of formation of end products. Raw materials are burned in calorimeter with paraffin oil as fuel, the heat of combustion of which is determined very accurately. Results obtained directly by this method are very satisfactory. Specific heat of components must be determined in order to calculate results for room temperature or temperatures in cement kiln.—A. E. BEITLICH

Influence of fineness on strength of portland cement. HANS KUHL. *Zement* (Germany), June, 1930, V. 19, No. 26 and 27, p. 604-8, 630-3.—Investigations were carried out to find best particle size of cement to obtain high strength properties. Finest cement portion does not give highest strength. Two cements were separated into 7 fractions from 10 to 60 μ by means of a Gonell air separator. Surface of particles was measured with formula derived by author. Rate of grinding and reaction of cement with water can be expressed by measured surface. Tensile and compressive strength tests with small briquets and cylinders were made after 3 and 7 days. Special care was taken for accurate weighing of cement, sand and water. Saturated gypsum solution was used to prevent quick setting of finest particles. Interesting relations were found between surface and "value-factor," which is calculated from strength data. High strength was obtained with uniform composition and particles of 30 μ size. Mixtures of certain fractions were made and their strength studied. Very fine ground cement can be improved by adding coarser portions within small range; it is not ecomomic to grind cement of uniform size to extreme fineness. Additions of 30 per cent of very coarse cement can be made without considerable loss in strength to a very fine but not uniform cement. The following conclusions were drawn: good cement can be made by grinding material to uniform size, but not too fine, or by grinding to very great fineness and adding coarse cement. Investigation will be continued.—A. E. BEITLICH

Substitution of compression for tension tests as standard tests of cements. *Revue des Mat. de Constr. et des Trav. Publ.*, July, 1930, No. 250, p. 253.—A superior Committee for Standardization was organized in France effective May 1, 1930. Its purpose is to coordinate methods of construction all over France and to bring about standardization. First steps to be taken

by the Committee are official recognition of new types of cement not covered by last standards, the substitution of compression for tension tests, the corresponding readjustment of various types of cement and their official nomenclature. Arguments in favor of substitution of compression tests are presented by R. Feret, Director of the Ponts et Chaussees Laboratory at Boulogne-sur-mer. The proposed limits of compressive strength for portland cement are: 100 kg. per sq. cm. at 7 days and 160 kg. per sq. cm. at 28 days. Strengths for high test portland cement are: 100 kg. per sq. cm. at 2 days; 250 kg. per sq. cm. at 7 days and 320 kg. per sq. cm. at 28 days.—M. A. CORBIN

Spanish specifications of February 25, 1930, for acceptance of cements on public works. *Le Ciment*, July, 1930, No. 7, p. 278.—Separate chapters are devoted to portland cement, high test cements (alumina cements), supercements, slag cements puzzuolan cements and Zumaya (natural) cements. The minimum tensile strength of 1:3 portland cement and standard sand mortar is specified as: 19 kg. per sq. cm. at 7 days (1 day in air, 6 in water) and 23.5 kg. per sq. cm. at 28 days (1 day in air, remainder in water). The minimum compressive strength is specified as: 190 kg. per sq. cm. at 7 days and 280 kg. per sq. cm. at 28 days. For alumina cements the tensile strength required at 24 hr. shall not be less than 28 kg. per sq. cm. and shall attain 60 kg. per sq. cm. at 3 days. The concrete strength tested on 20 cm. cubes of 840 liters gravel, 400 liters sand, 300 kg. cement and 130 liters water shall be greater than: 220 kg. per sq. cm. at 24 hr. and 230 kg. per sq. cm. at 28 days. The tensile strength of 1:3 mortar made with super-cement shall be: 25 kg. per sq. cm. at 3 days; 30 kg. per sq. cm. at 7 days; 38 kg. per sq. cm. at 28 days. The compressive strengths shall be: 250 kg. per sq. cm. at 3 days; 350 kg. per sq. cm. at 7 days and 450 kg. per sq. cm. at 28 days. The concrete strength shall be: 180 kg. per sq. cm. at 2 days and 230 kg. per sq. cm. at 7 days.—M. A. CORBIN

The Spanish cement specifications. C. R. PLATZMANN. *Zement* (Germany), July, 1930, V. 19, No. 30, p. 701-2.—In Feb. 1930, great changes in Spanish standard cement specifications of 1919 were made. New directions include portland cement, high early strength portland cement, alumina cement, blast furnace slag cement, puzzuolan cement and Zumaya cement. Latter is kind of natural cement. Comparison of old and new specifications for chemical composition, fineness, setting time, soundness and strength properties are given.—A. E. BEITLICH

Roller bearings in the cement industry. BELANI. *Zement* (Germany), July, 1930, V. 19, No. 28, p. 654-7.—Author discusses different types of roller- and ball-bearings which can be used in cement industry. Economic point of view is given. Advantages of certain types for special purposes are described. —A. E. BEITLICH

Experiments with mixed cements consisting of high alumina cement and ground basic blast furnace slag. WILLIAM PETRY. *Zement* (Germany), July, 1930, V. 19, No. 29, p. 674-8.—Mixtures were made from high alumina cement "Alca" and blast furnace slag in following proportions: cement : slag : sand, 1:0:3; ¾:¼:3; ½:½:3, and ¼:¾:3. Compressive and tensile strength tests were made after 3, 7 and 28 days. Results showed that no great decrease in tensile strength takes place when one half of cement is replaced by slag. This mixture gives compressive strength which is much higher than strength which is demanded by standard specifications. Not more than half of cement should be replaced by slag. Additional tests were carried out in order to investigate action of different salt solutions, such as magnesium sulphate (5 per cent), sodium sulphate (4 per cent), sulphuric acid (1 per cent), acetic acid (1 per cent) and ammonium sulphate (5 per cent). General appearance and tensile strength after 15 months storage of test pieces, which were immersed in these solutions, showed that action of first

four solutions is considerable lower when one half of cement is replaced by slag. Only ammonium sulphate solution produced great deterioration in every case.—A. E. BEITLICH

The heat in the finishing mill and the setting time of cements. HEISER. *Zement* (Germany), July, 1930, V. 19, No. 29, p. 679-80.—Discussion of research work done by K. Koyanagi (*Zement* (Germany), May 1930, V. 19, No. 20.) concerning influence of heat during grinding process of cement on its setting time. Based on own investigations author comes in general to different conclusions and claims that quick setting cement is not necessarily produced when temperature in finishing mill is very high. With temperatures up to 100° C. he still obtained normal setting clinker.—A. E. BEITLICH

The sealing of rotary kilns, dryers and similar equipment. ERICH SCHIRM. *Zement* (Germany), June-July, 1930, V. 19, No. 23, 24, 26 and 27, p. 545-8, 570-2, 618-20, 642-4.—Several different types of seals are described. Discussion shows which types are suitable for special purposes. All advantages and disadvantages are illustrated and directions are given to prevent excessive wear. Seventeen photographs are reproduced.—A. E. BEITLICH

Experiments with a compeb mill using special steel fillings. OTTO FREY and MOSCHITZ. *Zement* (Germany), July, 1930, V. 19, No. 28, p. 652-4. —Experiments were made with a three-compartment compeb mill to increase efficiency by studying best combination and weight of various steel fillings with different shapes. Best results were obtained with filling of steel balls in first compartment, special patented "Kordt-balls" in second and steel cubes with small steel balls in third compartment. With usual filling of steel balls in first and second and "Cylpeps" in third compartment 12,500 kg. material was ground in one hour to fineness of 2.5-4 per cent residue on 4900 mesh sieve. With new filling 15,000 kg. was ground in one hour to fineness of 0.5 per cent residue on 4900 mesh sieve. Power consumption was 540 H. P. instead of 520 H. P. Material was clinker for production of special high early strength cement.—A. E. BEITLICH

Scientific research. *Structural Engineer* (England), August, 1930, V. 8, No. 8, p. 302-3.—The use of "hot cement," meaning cement which still retains much of frictional heat of grinding, has been found by Building Research Department (Bulletin No. 7) to have unimportant effects when used "under conditions likely to be found in a job." Tests reported in Building Research Special Report No. 15 are said to prove that coal residues are definitely unsuitable for use as aggregates in concrete reinforced by or in contact with steel. Sulphur contents of materials are considered to be prime cause of corrosion, and porosity and permeability to be of minor significance. —V. P. JENSEN

Production of Norwegian cement industry. *Zement*, (Germany) May, 1930, V. 19, No. 22, p. 525.—Data are given which show progress in production of cement in Norway during last 20 years.—A. E. BEITLICH

Studies of the behavior of cement mortars in hot water. Experiments made at the laboratory for testing materials of the Technische Hochschule in Stuttgart (Germany) during the years 1928 and 1929. "Versuche ueber das Verhalten von Zement-moertel in heissem Wasser." OTTO GRAF. German committee for reinforced concrete, publication No. 62. Willhem Ernst und Sohn, Berlin 1930, R. M. 4.30. Reviewed in *Der Bauingenieur* (Germany), June 1930, V. 11, No. 24, p. 427.—Book describes investigations of behavior of mortars exposed to water of different temperatures and under different conditions. Four kinds of cement, two

portland cements, one high early strength portland cement and one high alumina cement were tested at 20, 50 and 90° C. Compressive and tensile strength of 1:3 and 1:6 mixtures were studied. Behavior of cements was very different. Loss in strength is discussed. Steam of 50° C. has improving effect on samples while water storage at 50° C. reduces strength. Author derives theory about setting of cement and its solution process.—A. E. BEITLICH

Burning of cement on a chain grate. ERICH SCHIRM. *Tonind. Ztg.* (Germany), July 24, 1930, V. 54, No. 59, p. 978-81.—The application of chain grate principle to burning cement and lime to reduce the fuel requirements. Trouble is that clinker sinters to the grate and Lellep has proposed (German pat. 466,298) to use the chain grate for calcining and rotary kiln for clinkering. Result is much saving in space. It is claimed that the chain grate makes effective dust and heat remover from the gases. Average requirement of less than 2000 B. T. U. per lb. of clinker is claimed.—F. O. ANDEREGG

Apparatus for laboratory control of fineness of cements and description of several experiments carried out with this equipment. A. H. M. ANDREASEN and J. J. V. LUNDBERG. *Zement* (Germany), July, 1930, V. 19, No. 30-31, p. 698-701,7 25-7.—Description of apparatus to determine fineness of cement in which sample is mixed with liquid media. Ethylene-glycol was found to be best liquid for this purpose which gives suspension free of coagulation without peptonizator. Its specific gravity at 20° C. is 1.112 and its viscosity at 20° C. is 0.214 which covers correct range for particles from 5 to 100 μ. Experiments were carried out with (1) two portland cements, (2) one blast furnace slag cement, (3) one French high alumina cement, (4) calcined magnesite and (5) stucco. Apparatus allows continuous taking of samples and gives results of high accuracy. Blast furnace slag cement shows some tendency to coagulate but gives good results. Strong coagulation was found in case of alumina cement which could be prevented by addition of cobalti chloride (0.005 mol per liter). Magnesite gave fairly good results in pure glycol. Mixture of 50 volume per cent absolute alcohol and 50 volume per cent glycol with specific gravity of 0.964 and viscosity of 0.0497 at 20° C. and an addition of potassium citrate (0.005 mol per liter) was used for stucco. Tables showing results obtained and several references are given.—A. E. BEITLICH

New means for protection of concrete against acids and aggressive waters. *Chemiker Zeitung* (Germany), Apr., 1930, V. 54, No. 27, p. 259-60. —Description of properties of number of new water-glass cements. It is claimed that they are acid proof and can be manufactured with quick and slow setting properties. Hardening is not influenced by any weather conditions and can also take place, when no air is present. Good cementing material for bricks of acidproof tanks, acid towers and sewage pipes. Temperatures up to 800° C. do not affect cement. It can be used with advantage for linings of concrete structures and foundations exposed to acid sprays or drops. —A. E. BEITLICH

Obtaining finely ground raw materials for cement. J. GUERIN. *Ciment.* (France), July, 1930, V. 35, No. 7, p. 262-6.—Rapid recirculation combined with filtration gives best results.—F. O. ANDEREGG

The reliability and predictive value of short time tension tests of portland cement. H. WALTER LEAVITT and JOHN W. GOWEN. *Maine Technology Exp. Sta. Bull. No. 25,* June, 1930, V. 16, No. 2, 8 pages.—Increased use of portland cement and advent of so-called super-cements have

caused highway engineers to demand more rapid tests for mortar strength than formerly. It has been shown that 7-day tensile strength tests will give fairly accurate prediction of 28-day tensile strength of portland cement. (c. f. "The prediction of 28-day tensile strength of portland cements from their 7-day strengths" by John W. Gowen and H. Walter Leavitt, Bulletin No. 15, pp. 1-14, May, 1926. Also "Relation of 7-day to 28-day compressive strength of mortar and concrete" by John W. Gowen and H. Walter Leavitt, Bulletin No. 17, pp. 1-13, Maine Technology Experiment Station, October, 1926.) This paper gives data on 150 samples of portland cement of many different brands as received at the Testing Laboratory for the latter half of the year 1929. It was shown that the variability of any briquet from a series of three made from the same batch does not change much either in a 1-day, 3-day, 7-day or 28-day tests. This conclusion was derived by a statistical study of the data in hand. The predictive value of the 1-day, 3-day and 7-day tests was determined and the following coefficients derived: 1-day, 3-day and 7-day tension vs. 28-day tension give correlation coefficients of 0.322 ± 0.049, 0.395 ± 0.047 and 0.734 ± 0.020 respectively. Previous study in 1926 showed 7-day tension vs. 28-day tension giving a correlation coefficient 0.654 ± 0.015. Higher coefficient obtained in 1930 is partly explained by better quality of cements now on market and partly by better technique in methods of testing. The conclusions were: (1) Inter-relation of one briquet with another in 1-day, 3-day, 7-day, and 28-day tests are relatively high. (2) The 7-day tension tests give much more reliable prediction of 28-day tension tests than 1-day or 3-day tension tests. (3) Data should be produced to show superiority of any other age period over 7-day period before discarding old preliminary test age (7-day) period.—H. WALTER LEAVITT

New 2,000-bbl. wet-process cement plant nears completion in Southwest. *Pit and Quarry*, July 2, 1930, V. 20, No. 7, p. 47-50 and 54.—New plant of American Portland Cement Co. at Foreman, Ark., is straight-line layout with provision for doubling capacity. Rock is chalky limestone of uniform quality. Quarry cars will be hauled by locomotives to trestle over hopper. Rock will be crushed to ½-in. size and coveyed to steel bins or diverted to open storage. Clay will be obtained from neighboring pit. Clay-slip from concrete washmill will be pumped to steel storage tank and thence it will pass to raw grinding tube-mill with rock. Ground material will pass to agitated slurry slump, and thence to storage tanks equipped with agitators. From these tanks, slurry will be pumped to correcting basins and thence to kiln feeder. Kiln is 11 ft. 3 in. in diam. and 300 ft. long, electrically welded throughout, with four tires. Clinker will pass to a vertical cooler with sloping shelves and sprays of water. Cement will be pumped pneumatically to 12 storage silos of a combined capacity of 100,000 bbl., from whence it will be pumped to packhouse containing three packers. 16 illustrations.—A. J. HOSKIN

Cement plant's capacity can be trebled without stopping operation. *Pit and Quarry*, July 2, 1930, V. 20, No. 7, p. 51-53.—At Rosedale, N. Y., plant of Century Cement Corp., rock occurs in two beds separated by 15 ft. of impure limestone. Excavation is by room-and-pillar underground method, in each bed. Mined rock from two sets of workings is kept separate and is hauled to trestle above eight vertical kilns which utilize buckwheat anthracite fuel. Clinker is drawn, hand-picked, crushed to ¾-in., elevated, and deposited in three bins—one bin for the burned light-colored rock, one for the dark rock, and one for occasional needs. The clinker is next taken by an aerial tramway, to top of finishing plant. Tram buckets are loaded and unloaded automatically. Clinker is stored in three steel bins, then withdrawn, weighed, and fed to grinding mill. Pulverized product is air-collected into bin over final grinding tube-mill. Natural cement goes to four storage silos and

enroute to packers, it passes vibrating screens, the oversize returning to the finish tube-mill.—A. J. HOSKIN

Arkansas' only producing cement plant embodies simplicity and safety. W. E. TRAUFFER. *Pit and Quarry*, July 2, 1930, v. 20, No. 7, p. 35-46, 47 illustrations.—New plant of Arkansas Portland Cement Co. at Okay started operation last fall. Mill construction began in Nov., 1928. Crushing began June 18, 1929, and first shipment of finished cement was made Oct. 2, 1929, 125 carloads. There are only four principal pieces of machinery in the plant: a No. 60 swinghammer crusher, two Compeb mills, and 300-ft. rotary kiln. Raw materials are from bed of Austin chalk and overlying bed of argillaceous loam. Blasted rock and clay are hauled to plant in steel side-dump cars. Quarry-haulage system is merely temporary, and is, eventually, to be converted into electric, central-tower-control system. Single crusher can reduce from 35-in. to 1-in. Stone and clay are excavated on alternate days. Entire storage space is divided into four compartments with concrete walls. Gypsum is unloaded from box cars by power scraper to conveyor directly to storage. Silica sand, required to maintain correct SiO_2 content, is unloaded from gondola cars by same power scraper and conveyor. Rock, clay and sand discharge through plate feeder to pugmill. This, discharges to 3-compartment raw-grinding mill which uses steel balls, and resulting slurry is handled by pumps to any of six 400-bbl. concrete batch tanks, equipped with mechanical and air agitation. Slurry, in proper proportion, is pumped into concrete mixing tank, thence to slurry-storage tank, also equipped with mechanical and air agitator. Slurry moisture is maintained at 47 per cent. Kiln is 11½ ft. in diam. and 300 ft. long, and uses natural gas fuel. Slurry filter is being installed to increase kiln's capacity from 2,400 bbl. to 3,000 bbl. per day. Slurry cake will be discharged from filter to belt conveyor discharging to feeder, to kiln. Clinker drops from the kiln to steel dragchain, discharging to iron bucket elevator, that handles clinker to storage. Finish mill is 3-compartment, compeb mill with a peripheral screen and pick-up pipes between the first and second compartments. Ground cement is fed to pneumatic pumps by which cement is carried through 5-in. line to any of 12 storage silos, having total capacity of 230,000 bbl. Exbiners, screw conveyors and bucket elevators deliver cement to packhouse. Article gives details of power and transmission equipment, and describes laboratories, offices, shops and village with water supply and sewage disposal.—A. J. HOSKIN

PROPERTIES OF CONCRETE

Protecting reinforced concrete against corrosive solutions. HANS VÄTH. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 61, p. 1010-1, No. 62, p. 1026-8.—Foundation for a coke plant was built of aluminous cement (Alca) in contact with $CaSO_4$ soln. Above ground it began to deteriorate within 6 months. Test specimens proportioned 1:3, in water were in perfect shape and showed increased strength after 142 days, but specimens of mixes 1:6 and 1:10 suffered cracking and loss of strength. When placed in 10 per cent $MgSO_4$ soln. specimens of 1:3 and 1:6 seemed in good condition but were weaker after 131 days. The 1:10 specimens lost no strength, although they became white. When stored in ground water from the coke plant the 1:3 and 1:6 specimens appeared in good shape but lost strength, while the 1:10 deteriorated badly. When planted at the building a 1:5.7 specimen 4 by 4 by 22 in. bar cracked badly in 120 days. Similar specimens of high early strength portland and of slag cements were in good shape at end of same period.—F. O. ANDEREGG

The structure of mortar and concrete. "Der Aufbau des Moertels und des Betons." OTTO GRAF. 3rd edition, 1930, Julius Springer, Berlin W9 (Germany), R. M. 17.50. Reviewed in *Zement* (Germany), June, 1930, V. 19, No. 25, p. 596.—This enlarged edition deals with compressive strength, flexural strength, weather resistance, water permeability, shrinkage and expansion of cements and mortars. Special consideration is given to concrete made with volcanic slags and pumice. Experiments with mortars of different compositions are described. Directions are given for concrete control on the job.—A. E. BEITLICH

The concrete brick. "Der Betonwerkstein." 1930. Editor: Bund der Deutschen Betonwerke E. V. Berlin NW6 (Germany). Reviewed in *Zement* (Germany), June, 1930, V. 19, No. 24, p. 572—Properties of concrete bricks and their application in practice are described. Directions are given for their valuation.—A. E. BEITLICH

The yield of certain concrete mixes. A. G. DARWIN. *Concrete*, July, 1930, V. 37, No. 1, p. 29.—Since cement is usually most costly ingredient of concrete mix, it is desirable to so proportion aggregate constituents that for given required strength, range of workability and other factors, the minimum cement factor obtains. Following are some general conclusions that may be drawn from yield study of Moscow crushed basalt rock and Spokane sand concrete, but only under laboratory conditions. (1) For given water-cement ratios of 0.85, 0.90, and 0.95, yield decreases at nearly uniform rate with increase in dry-rodded proportion of sand to rock. (2) For given proportion of sand to rock by dry-rodded volumes, yield increases as water-cement ratio increases. (3) For a given proportion of sand to rock the yield increases as strength decreases.—C. BACHMANN

Miscellaneous concrete construction practices. K. GOSLICH. *Zement* (Germany), June, 1930, V. 19, No. 24, p. 567-8.—Disintegration of concrete is caused by water containing sulphuric acid or sulphates, especially the magnesium salt. New experiments seem to prove erroneous the conclusion that cements which are high in lime are more easily attacked by such waters. Most important factor is density of concrete which can be obtained by expensive rich mixtures or by carefully proportioned aggregates. Concrete pieces which are partly immersed in water show greatest corrosion at line between water and air. Capillarity of concrete and concentration of salts by evaporation are to blame for this effect. Paraffin and asphalt or mixtures of asphalt and tar are suggested for protection of concrete pipes.—A. E. BEITLICH

Solution phenomena of concrete. RICHARD GRUN. *Der Bauingenieur* (Germany), June, 1930, V. 11, No. 26, p. 451-4.—Solution of concrete can take place in two different ways, (1) by water which is poor in carbon dioxide by direct solution of lime, or (2) by water rich in carbon dioxide. In second case, lime is converted to calcium carbonate, then to soluble calcium bicarbonate and finally back to calcium carbonate which appears in voids and on surface of concrete. These reactions cause loss in strength and deterioration of concrete. Such action can be prevented by making dense concrete which does not allow penetration of water. Use of cement which is low in lime and addition of certain admixtures are suggested for concrete protection.—A. E. BEITLICH

Winter-laid 10,000-lb. concrete on Chicago viaduct. ARTHUR R. LORD. *Eng. News Record*, Aug. 14, 1930, V. 105, No. 7, p. 258. During the construction of new bascule bridge across Chicago River at Wabash Ave. in Chicago, a

large area of concrete deck slab and several supporting columns were removed from Wacker Drive. Chunks of concrete from the debris were sawed into 3x3-in. cubes for testing. Tests showed an average strength for cubes of 11,400 lb. per sq. in. Equivalent cylinder strength is considered to be about 85 per cent of this value. Cylinder tests at 28 days showed a strength of 2,620 lb. per sq. in. On this basis, strengths at 46 months are over three times those at 28 days, and over 2½ times those that would be expected at 28 days in summer weather for this mix and water-cement ratio. This concrete was made with seven sacks of cement, 14 cu. ft. of Beloit sand, and 22 cu. ft. of crushed limestone. Water-cement ratio was 6.5 gal. of water per sack of cement. During placing, the temperature of air above the slab varied from zero to slightly above freezing. All materials were heated to give temperatures in fresh concrete of about 80° F. at mixer, about 60° at forms and about 35° to 40° at upper surface of slab by the time it had hardened sufficiently for workmen to cover it with straw. Air beneath the slab, heated by salamanders, varied from 55° to 85° F. during curing period of seven days. Under the straw, temperatures at the surface of the slab were above 50° F. during the week.—D. E. LARSON

Shrinkage of concrete as a factor in compressive steel stress. (See ENGINEERING DESIGN—Miscellaneous.)

Dock and harbor construction in British colonies. (See ENGINEERING DESIGN—Miscellaneous.)

Vibrated concrete. (See FIELD CONSTRUCTION—Miscellaneous.)

Effect of soft particles of coarse aggregate on strength and durability of concrete. (See MATERIALS—Aggregates.)

Studies of behavior of cement mortars in hot water. (See MATERIALS—Cement.)

ENGINEERING DESIGN

BRIDGES

International contest for construction of bridge over Lake Maelar in Stockholm (Sweden). *Zement* (Germany), July, 1930, V. 19, No. 30, p. 712-4.—First prize was given to plans by Prof. F. von Emperger, Vienna for reinforced concrete bridge consisting of two arches with span of 198 m. each. Roadway is 17.50 m. wide with two 2.25 m. wide sidewalks and consists of 5 cm. thick asphalt slabs. Dimensions of structure and details of reinforcements are given.—A. E. BEITLICH

Concrete arch Mississippi bridge at Minneapolis. *Eng. New Record*, July 10, 1930, V. 105, No. 2, p. 49-50.—Two reinforced-concrete rib-arch spans of 265½ ft. in the clear form the channel portion of the new Cedar Ave. bridge across Mississippi River at Minneapolis, Minn. They have rise of 90.5 ft. and are located on tangent of 611 ft., crossing river on skew. Three flanking arch spans on west side and two on east side are of 93 ft. span and are on curves of 5 deg. 41 min. and 5 deg. 18 min. respectively. All arch spans have two ribs 12 ft. wide and 24 ft. apart in clear, with thickness for main spans of 3.5 ft. at crown and 7.5 ft. at haunches. A three-center curve having radii of 154 and 48.5 ft. is used for intrados, and a curve having a single radius of 167.5 ft. is used for extrados. Each main span rib is reinforced with five steel ribs 31½ in. deep at crown. Chords are composed of pairs of angles 3½ x 3½ x ⅜ in. and web members of smaller angles. Each rib in side spans

is reinforced with sixteen longitudinal bars top and bottom tied together with loop bars 4 ft. apart. Footings for three river piers are 30 x 54 ft. At east and center piers they rest on sandstone bedrock while west pier footings are supported on 180 piles 32 to 42 ft. long driven into gravel formations and spaced 3 ft. c. to c. both ways. Transverse spandrel walls practically full width of arch ribs, and spaced 13 ft. 4½ in. c. to c. in the main spans and 12 ft. in the flanking spans, have cantilever brackets for sidewalls and carry pairs of transverse 24-in. I-beams which support 12-in. reinforced-concrete deck slabs.—D. E. LARSON

Design of road bridges. C. S. CHETTOE. *Roads and Rd. Constr.* (England), June 2, 1930, V. 8, No. 90, p. 181.—Culverts. Except where there are rock foundations, culverts usually have inverts. Total foundation pressure is equal to vertical component of forces acting on culvert top, plus weight of culvert and if necessary, water, when it is running full. On mass concrete or rock foundation, pressure would be either concentrated under walls, or distributed along whole base, with maximum at centre, according to whether moments due to forces caused convexity or concavity upwards, in invert. On yielding clay, pressure tends to become uniform for all conditions of loading. Culvert is designed as two fixed elastic arches with abutments at centre of top and invert, where no angular displacement is assumed. Horizontal thrust on invert is equal to actual forces on one side of culvert less horizontal thrust at crown. Shear at sharp bends may be appreciable. Ideal shape is a circle. No allowance need be made for temperature shrinkage, or rib shortening. Formulae are given for the necessary bending moments. Rectangular culvert is most suitable for spans up to 8 or 10 ft. on account of simplicity of shuttering and steel fixing, although it is not most economical shape for material. Large culverts are usually semi-elliptical with straight or slightly rounded inverts, and sometimes, extended invert ends. For culverts passing beneath deep bank, on compressible soil, special longitudinal reinforcement may be required either in deck and invert, or at four corners, to take up positive bending moments. Vertical reinforcement may need to be increased to take shear.—JOHN E. ADAMS

Design of road bridges. C. S. CHETTOE. *Roads and Rd. Constr.* (England), July 1, 1930, V. 8, No. 91, p. 217.—Abutments: If filling is not to be completed until after decking is in position, final stresses only need be considered; otherwise abutment wall should be designed to act as retaining wall during construction, and to take final live and dead loads in addition. Downward forces on abutment are weight W' and reaction from girders, W''. The horizontal forces will be P, due to earth pressure and surcharge and $\pm W''$ x m, where m is the coefficient of friction between girder and bearing plate. Abutments should be able to withstand any movement due to lengthening or shortening of girders. Mass concrete girders should have thickness of 0.35 x height at base and be relatively thicker at top than ordinary retaining walls. For spans up to 20 ft. rolled steel joists are allowed to rest directly on smooth concrete without provision for expansion. Bearing plates are usually provided for built-up girders, up to 40 ft., girders from 40 to 60 ft. span usually are carried on cast iron or steel plain bearings arranged for sliding at one. When rein. conc. girders are used no expansion bearings are necessary up to 35 ft. span, particularly if carried on piles. Spans up to 50 ft. may have felt bearings, while larger spans have two outer and two inner plates of steel and copper respectively. In lighter, reinforced concrete abutments having counterforts and sill beams, which take the decking beams, the wall prevents sill beams from deflecting and taking much of load. Curtain walls which span horizontally, are designed for moments of $\frac{pl^2}{12}$ at counterforts, and $\frac{pl^2}{24}$ at mid

span. The end spans are designed for $\frac{pl^2}{10}$. Distribution steel must be provided at sill and junction with foundation, to avoid cracks due to reverse moments. Counterforts are designed as ordinary retaining walls and should be checked for safety before and after decking is on, and for full live loads. For arch abutments, of mass concrete, final thrust should approximately bisect bearing surface and be normal to it. The top of arch abutment must never be finished off horizontally or it will have no shear resistance. Abutments may be cellular with a sill beam. Active earth pressure, only, is allowed for except where abutments are carried against hard virgin soil, or rock. In poor ground, short cantilevered end-spans are economical. Piers: In case of adjacent arch spans, resultant thrust at base should lie within middle third, worst case being when one span is fully loaded, and other unloaded. If A is free horizontal deflection due to maximum difference of horizontal thrust at pier top, and B horizontal movement of springings if free to slide on pier top, due to the same force, then the horizontal force taken by pier will be approximately $\frac{B}{A+B}$ and the rest will be taken by the arches.—JOHN E. ADAMS

Exhibition of new concrete bridges built in Bavaria (Germany). A. ZENNS. *Zement* (Germany), July, 1930, V. 19, No. 28, p. 660-3.—Design and construction of a number of reinforced concrete arch-, frame- and girder-bridges are shown by several photographs and drawings.—A. E. BEITLICH

Arch bridge near Echelsbach (Germany). (See FIELD CONSTRUCTION —Bridges.)

BUILDINGS

Tables for construction technique. "Bautechnische Tabellen." KARL-JOHANN HOCHREUTER. 1930, Gustav Winters, Bremen (Germany). Reviewed in *Zement* (Germany), July, 1930, V. 19, No. 30, p. 714.—Book contains tables for mathematical calculations of static and dynamic stresses in steel and reinforced concrete construction. Directions for erection of such structures are given. Tables are valuable aid to contractor and designer.—A. E. BEITLICH

Warehouse in Minden (Germany). *Zement* (Germany), June, 1930, V. 19, No. 23, p. 538-9.—A storage house with a capacity of 6000 tons wheat was built in reinforced concrete skeleton construction. Four floors, designed for live load of 2000 kg./m², were constructed with slab-girders. Eight bins, 13 m. high and 3.80 m. x 2.60 m. wide, are on north side. Grain elevators, mill equipment and offices are located in building.—A. E. BEITLICH

Tests on concrete frames simulate effects of earthquake action on buildings. *Eng. News Record*, July 10, 1930, V. 105, No. 2, p. 55.—This article, which is an abstract of paper read before recent international engineering congress in Japan by Mikishi Abe, consulting engineer of Tokyo, presents conclusions from tests made to study action of rectangular reinforced concrete frames under repeated horizontal load. In these tests, made at University of Illinois, periods of free or natural vibration of reinforced concrete frames were measured in order to compare them with known formulas. Four main points considered were: (1) Behavior of frames under horizontal loads repeated many thousand times. (2) Free vibration periods before and after cracking of frames. (3) Effect of construction joints upon horizontal deflection under repeated static horizontal loads. (4) Methods of improvement in design and construction. Mr. Abe stated that the results were in fair agreement with the analyses. Several of his conclusions are summarized as follows: (1) Observed stresses in frames were not increased by

repeated application at low loads. Under high loads, stresses were slightly increased, but intensity was well within computed values. (2) Periods of free vibration of test frames, measured by aid of motion photographs, were about 0.03 second, before any cracks had developed. After cracks had developed, periods were about 0.04 second. From these results, it is argued that there is small possibility of synchronizing natural vibrations of a building with vibrations of a great earthquake after the composing members have been cracked. (3) Horizontal deflections of frame with ordinary construction joints at feet of columns were almost equal to those of frame with no such joints, but were much larger than where dowels were used. (4) Use of large haunches at end of beam was apparently inadvisable because many small cracks appeared in haunches. Bending stresses at top of the column were much reduced, while stress at bottom was not reduced by use of large haunches. (5) Construction joint in beam is not objectionable if properly built although cracks due to vibration tend to follow joint.—D. E. LARSON

Design and construction of modern factory in reinforced concrete. A. W. LEGAT. *Structural Engineer* (England). May, 1930, V. 8, No. 5, p. 178-190.—Soap factory requiring 4066 cu. yds. of concrete is described in detail from selection of site and general dimensions to details of design, materials, and methods of construction. Building features reinforced concrete loading bays, milling bay including hoppers, and in general reinforced concrete for the entire building with two exceptions, first, where steel was used for roof trusses and materials storage tanks, and second, where brick was employed for sleeper walls and temporary end paneling.—V. P. JENSEN

New structures of paper plant in Scheufelen near Oberlenningen i. Wuert. (Germany). A. WEGENAST. *Zement* (Germany), June, 1930, V. 19, No. 24, p. 565-7.—Several new buildings were erected in enlarging program of big paper plant. One building which serves as connecting link between old buildings is 23.65 m. long, 11.75 m. wide and 31 m. high. It contains main stairway, washrooms and several rooms for general purposes. Entire building and stairway is constructed of reinforced concrete. Main building, which houses machinery and material storage is 5-story structure with two rows of center columns spaced 5.50 m. apart. Connecting beams have span of 7.50 and 9.65 m. High early strength portland cement was used for their construction. Ceilings are "Remy" ceilings designed for live load of 1000 kg./m.^{2} Third building shows interesting ceiling construction. Area of 2960 m^2 is covered with only eight columns. High quality iron-portland cement was used. Transformer building is 49.50 m. long, 6.00 m. wide and 4.30 m. high.—A. E. BEITLICH

DAM

Cobble Mountain dam of Springfield, Mass. JAMES B. PORTER. *Eng. and Con.*, July, 1930, p. 255-257.—Highest hydraulic fill dam in the world will make use of concrete cut-off walls in bottom of dam. These walls extend up slopes on each side. Concrete arched wall 14 ft. wide on top, with vertical upstream face and ¼ to 1 downstream face, and with 100 ft. radius is to be at downstream toe of hydraulic fill dam. Spillway will care for 400 second feet per square mile of watershed. Reinforced concrete arch bridge will be built over spillway and will carry highway that goes over top of dam.—N. H. ROY

MISCELLANEOUS

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete.* June, 1930, p. 45-47.—Design of long columns, h/R greater than 50, according to A. C. I. specifications is greatly simplified by use of a nomographic chart containing all necessary quantities.—N. H. ROY

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete*, July, 1930, V. 37, No. 1, p. 43-46.—Prof. Griffith has constructed charts to simplify design of tied columns according to A. C. I. Code. Three quantities are given in charts, area of longitudinal steel, total column load, and size of square columns for 2000 and 3000 pound concrete. Comparison is made with Joint Committee requirements. A. C. I. Code is the more liberal of the two. —N. H. ROY

Method of calculating strength of steel beams encased in concrete. O. H. MENGERING. *Concrete*, June, 1930, V. 36, No. 6, pp. 35-37.—Rational method of analysis of composite steel and concrete beam sections is presented. Author gives methods of obtaining areas and statical moments and location of neutral axis of compound sections. Expressions for K, kd, and b, width of member, are set up for balanced section. Sufficient material, concrete or steel, is supplied to balance tensile steel area. Expressions for moments of inertia of compound and balanced sections are also given. Likewise equations for moment of inertia and width of stem of tee-beams are given. Shear, bond, deflections and resisting moments are discussed and typical problems solved. Width of beams is found to vary with grade of concrete and is inversely proportional to value of n, ratio of moduli of elasticity of steel and concrete. For the section chosen, the resisting moment was increased about 13 per cent by encasing in concrete. The center deflection of simply supported beam of the section used was from 36 to 27.5 per cent less for the concrete encased beam than for steel section alone.—N. H. ROY

Shrinkage of concrete as a factor in compressive steel stress. *Eng. and Con.*, June, 1930, p. 234-236.—Prof. Maney's discussion and conclusions, as presented to Western Society of Engineers April 7, are summarized and two general conclusions drawn regarding tremendous increase in compressive steel stress in a beam under sustained loading over period of two months. Plastic flow is negligible factor. Shrinkage is controlling factor. Amount and effect of shrinkage are predictable. A law regarding shrinkage has been evaluated for benefit of designers. Amount of shrinkage was found to be controlled by percentage by volume of cement paste and quality or water-cement ratio of paste. Data from experiments is given partly by curves. Measured stresses in compressive steel after 40 days of continuous loading were found to be approximately 2½ times the calculated stresses. Deformations in concrete at compressive surface were very high. Flow of concrete around reinforcing rods took place without damage to bond. A. C. I. Column Code and possibilities for its improvement are discussed at length. Specifications nf_c for compressive steel holds only for load changes; for time loading it is not sufficient. For ordinary case nf_c is only a small part of total compressive steel stress. It is predicted that compressive steel with high elastic limit and controlled shrinkage will feature changes about to take place in reinforced-concrete design.—N. H. ROY

Steel beams much stronger when encased in concrete. F. A. RANDALL. (Report of Committee of the Western Society of Engineers), *Concrete*, June, 1930, V. 36, No. 6, p. 34.—Tests made at plant of American Bridge Co., at Gary, Indiana, on floor framing encased in concrete, were completed and reported upon on May 12. Floor system was described in *Concrete*, Nov., 1929, p. 24. Deflections and deformations were measured on steel members, which were then encased in concrete and readings taken again after 28 days. Loads up to twice dead plus design live loads were applied. Test results were compared with usual design computations. Tensile stress in bottom flange of steel beams was greater than compressive stress in upper flange, due to concrete in upper part of member taking compressive stress,

whereas concrete in lower part of member probably did not take any tensile stress. Thus neutral axis of composite member was in effect raised and tensile stress was lower than it would have been in an unencased member. Measured stresses in tension flanges of fully loaded beams averaged 56 per cent of so-called computed stresses for cinder concrete panels and 57 per cent for gravel concrete panels. "Computed stress" denotes theoretical stress obtained by usual design methods, considering that steel section carries entire load. Committee recommends working stress of 24,000 lbs. per sq. in. be allowed on steel section when it is encased in manner similar to that of tests and using materials of like quality on I-beams and girders up to 24 in. in depth. Full committee report is contained in the June Journal of the Western Society of Engineers.—N. H. ROY

The steel requirement for reinforced concrete beams. KURT BERNHARD. *Zement* (Germany), June-July, 1930, V. 19, No. 25, 27, 28, 29, p. 591-5, 637-40, 663-6, 688-90.—Author illustrates influence of "steel-coefficient" for calculation of reinforced concrete beams as this was done for concrete slabs in previous article (*Zement* (Germany), April-May, 1930, V. 19, No. 16-20, p. 379-83, 405-9, 447-52, 474-8). Examples of practice showing steel requirements under all possible conditions of stresses are cited. Formulas and calculations are given for single beams and panel construction.—A. E. BEITLICH

Water power stations. "Wasserkraftanlagen." FELIX BUNDSCHU. 1929, 130 pp. Walter de Gruyter, Berlin, R. M. 3.00. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), April, 1930, V. 82, No. 17-18, p. 156.—Book gives (1) general directions for design and construction of water power stations, (2) describes locks and breakwater and (3) water pressure pipe construction and laying.—A. E. BEITLICH

Calculation of masses in reinforced concrete construction on theoretical basis. "Die Massenberechnung im Eisenbetonbau auf theoretischer Grundlage." FRITZ BUECHI. 1929, 139 pp. Wilhelm Ernst und Sohn, Berlin, R. M. 5.60. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), May, 1930, V. 82, No. 21-22, p. 188.—Book discusses analyses of calculations for masses in concrete construction and gives formulas for calculation of all kinds of cross-sections. Several examples of practice are cited and numerous tables are given.—A. E. BEITLICH

Pile driving calculations with notes on driving forces and ground resistance. A. HILEY. *Structural Engineer* (England). July, 1930, Vol. 8, No. 7, p. 246-259 and August, 1930, V. 8, No. 8, p. 278-288.—A comprehensive treatment of pile driving calculations giving both theoretical analyses and experimental data for various soil conditions, types of hammer, and types of pile.—V. P. JENSEN

Experimental structural engineering. H. W. COULTAS. *Structural Engineer* (England), August, 1930, V. 8, No. 8, p. 290-301.—Theoretical discussion includes an historical review bringing out laws of general work and virtual work, Mohr's equations, and Maxwell's theorem of reciprocal deflections. Solution of indeterminate structures by means of models is described and Professor Begg's method and apparatus are discussed in detail.—V. P. JENSEN

Design of River Des Peres drainage channel, St. Louis, Mo. W. W. HORNER. *Eng. News Record*, July 24, 1930, V. 105, No. 4, p. 124-128.—Four of 13 miles of River des Peres Drainage Canal now under construction in St. Louis, Mo., will consist of reinforced concrete arch conduit. Of this

amount, 10,000 ft. will consist of twin arches each 29 ft. wide and remainder of single arches 32 ft. wide. In designing the arches, stress analyses were made for both completely fixed and completely hinged end conditions and the actual detailing was on the basis of stresses less than maximum in either case. Sections were designed for earth backfills of 5, 10, 15, and 20 ft. over the crown, and for three different bearing values under footings. Specified working stresses were 800 lb. per sq. in. for concrete and 20,000 lb. per sq. in for steel. Six sacks of cement were used for each cu. yd. with not more than 55 lb. of total water per sack of cement. Aggregates were varied to secure most workable concrete consistent with good practice. Materials were measured by weight and mixing time of 1½ minutes was required. Steel forms were specified and were required to remain in place until concrete had attained compressive strength of at least 1,200 lb. per sq. in. To determine this strength, cylinders were cast at time of pouring and cured under conditions similar to those of concrete in forms.—D. E. LARSON

Dock and harbor construction in British colonies. MAJOR JOHNSTONE-TAYLOR. *Concrete*, July, 1930, V. 37, No. 1, p. 23.—These notes are principally concerned with the use of reinforced concrete as an alternative to timber for jetty and bank protection work in a British Colonial development in Asiatic waters. On Whang Poo River, Shanghai, there are comprehensive works consisting of reinforced concrete skeleton wharf supported on piles of same material and reinforced concrete quay wall of buttressed class forming part of same works. Wharf is 174 ft. wide and 1160 ft. long. Piers are 15 in. square and spaced 15 ft. apart in each direction. Each pile in turn is supported on a group of 14 in. square piles. Beams run across the wharf from side to side over each row of piers and they support a system of cross beams spaced 5 ft. on centers. Bases of piers which are pile caps are connected both ways by horizontal members. Piles were molded in a horizontal position. Driving was effected without use of a steel helmet. A thin piece of pine was placed directly on the pile head, above which was a ¾ in. steel plate surmounted by 4 in. of hardwood. Arrangements were made for the pile to run in guides between rubber covered rollers packed with thin pine battens.—C. BACHMANN

Nomogram for calculating columns without spiral reinforcing (Axial load). JORGE QUIJANA. *Ingenieria* (Mexico), Feb., 1930, p. 57.—A simple alignment or nomographic chart, one of a series of such charts, for use in reinforced concrete design and investigation, solving the relation $P = [A_c + (n - 1)A_s] f_c$. Left hand scale or line A_c, middle scale P and right hand scale A_s. Explanation of the use is given on the chart. All in metric units.—MILES N. CLAIR and C. G. CLAIR

Nomogram for calculating columns reinforced longitudinally and with spiral reinforcing. JORGE QUIJANO *Ingenieria* (Mexico), Feb., 1930, p. 73.—A simple alignment or nomographic chart, one of a series of such charts for use in reinforced concrete design and investigation, solving the relation $P = A_c f_c + n f_c p A$. Left hand scale or line D or column diameter, middle scale P, and right hand scale p. Explanation of use is given on the chart. All in metric units.—MILES N. CLAIR and C. G. CLAIR

ARCHITECTURAL DESIGN

Modern church structures by Prof. Dominikus Boehm. A. HOFF. *Zement* (Germany), July, 1930, V. 19, No. 27, p. 640-1.—Several church constructions by one of the leading German architects are described. Modern church design differs very much from earlier methods. Vaults of great spans are desired which can be made of reinforced concrete. Arch construction without centerings were used by architect in building new church in Neu-

Ulm (Germany). Excellent acoustical and light effects were obtained in this structure. Similar designs were used in construction of churches in Gross-Welzheim and Frielingsdorf where special decorative window construction provides indirect light. Mass concrete is being used for convent in Muenchen-Gladbach. Finish of surface adds to strong effects. Concrete is more suited than any other material to produce architectonical effects in modern church construction.—A. E. BEITLICH

New Concrete filteration plant. FREDERICK COLEMAN. *Architect Building News* (England), May 9, 1930, V. 123, No. 3203, p. 35.—The new concrete filtration plant of Metropolitan Water Board at Kempton, England, is a splendid example of artistic possibilities provided by massing of concrete. This ferro-concrete building is a monolith: column and beam, arch and wall become *one* stone, in which connecting steel rods link the most distant parts together. The possibilities of texture, of color inhering in the product make it a thing through which the designer can make his feelings flow. The more a building tends toward a mono-material building, the more nearly will perfect style reward an organic plan and ease of execution economize results. The more logical will the whole become. In all real art, structure and design form an inseparable unity. . . . The problem is to express in beautiful forms the type of construction thrust upon us by a utilitarian age.—REXFORD NEWCOMB

Concrete. DR. ERICH MENDELSOHN. *Architect Building News*, (England), May 23, 1930, V. 113, No. 3205, p. 671-72.—The pliability of concrete leads many an architect to continue in his old habits with the new material, and not only as regards architectural style, but also in his building technique. But to build in concrete involves a revolution in building technique. We build frames—that is, we concentrate the load on single points instead of distributing it over the wall as a whole. We pour in concrete instead of, as formerly, setting the single parts, bricks or stones, on top of each other. We assemble a building like a machine; mechanical accuracy replaces the skill of craftsmen and his individual subtleties. For our buildings we require elevators and spouting towers instead of mason's guilds and scaffolding; we calculate exactly beforehand every detail of the construction; we organize on a large scale instead of being contented with empirical attempts and occasional lucky results. This fundamental alteration in the methods of building is one of the causes of change in ideas on architectural form. The success of the new material consequently give the architect entirely new creative possibilities. The second quality of the new materials, steel and concrete, that of being able to live a life of their own, to exist as absolutely independent materials, gives the lie to those architects who see in them only an additional means of realizing dreams born of their excursions into a mummified past. We rejoice in the new materials because they impose on us obligations which, if we accept, set us free for new and original creations. We know that constructions in steel and reinforced concrete are very difficult to alter. That is why we consider the plan of the building as though it were a living being. We carefully avoid anything in the nature of a schematic design, because each plan has its own life, its own purpose, its own conditions of movement and transport, its own unique form. We reckon with it and with its form as something determined by its nature—and thus harmonize calculation and form, industrial purpose and architectural form. We settle in advance the sequence of separate parts of the building; we determine its rhythm, which is of more value to us than any decorative trimmings. We see in constructing a creative activity; in the creative construction, an architectural work of art. With our girders we span the greatest spaces. We light them by great glass surfaces between the elastic supports. By means of glass we get rid of inert matter and make the building light and transparent. We transfer the principles of bridge construction to building and by means of cantilevers raise certain parts above the earth, and cause them to float in the air.—REXFORD NEWCOMB

Lincoln Cathedral repairs. *Architect Building News* (England), April 18, 1930, V. 113, No. 3200, p. 499.—The reparation of Lincoln Cathedral has so far proceeded that the interior of the nave is at last freed from scaffoldings. The reinforced concrete ties, holding in the west front which was in danger of falling outwards, have been completed; as also, similar reinforcements to prevent the future spreading of the arches under central tower, and the buckling of the nave walls.—REXFORD NEWCOMB

Concrete church Jacobi in Stettin (Germany). (See FIELD CONSTRUCTION—Buildings.)

FIELD CONSTRUCTION

BRIDGES

Bridge over river Allien in Langeac (France). FRITZ EMPERGER. *Zement* (Germany), June, 1930, V. 19, No. 23, p. 537-8.—From spring 1927 to spring 1929 this reinforced concrete highway bridge was built. River is 2 m. deep and 20 m. wide during greater part of year, but rises about 6 m. and is about 140 m. wide during high water periods. Total structure is 140 m. long and consists of two concrete arches carried on two shore abutments and one river pier. Bridge crosses river at approximate angle of 71 degrees. Span of each arch is 68 m. and their height is 13.6 m. Arch ribs are 0.60 m. wide and from 1.20 m. to 1.60 m. high. Roadway is 5.20 m. wide and has two sidewalks for pedestrians 1.10 m. wide on each side. Floor system shows normal design; it is suspended from the arches. Concrete floor slabs are covered with layer of asphalt. One centering was used for both arches since they have same dimensions. Difficulties were encountered in making foundations for river pier.—A. E. BEITLICH

Arrangement and preparation of structural materials for construction of bridge over river Ammer near Echelsbach (Germany). FERDINAND DUELL. *Die Bautechnik* (Germany), Jan., 1930, V. 8, No. 1 and 4, p. 9-15, 52-5.—Aggregates were found in 4 km. distance from site of construction. Gravel, which had high loam content was washed in special plant. Water was pumped 80 m. high from river with high pressure pump which delivered 8 cu. meter of water per hour through pipe (52 mm. in diameter) to tank of 10 cu. meter capacity. Sand was transported by truck to storage bins. Aerial tramway carried materials to opposite side of river. Two concrete mixers of 350 and 500 liter capacity were used. Cement silo had room for 25 carloads. Laboratory for testing cement and concrete was erected near site. Distribution and placing of concrete during all stages of construction are described. (cf. *Der Bauingenieur* (Germany), June 1930, V. 11, No. 23, p. 391-406.)—A. E. BEITLICH

Reinforcing of bridge over the river Roeder between Goerlitz and Dresden (Germany). FRITZSCHE. *Die Bautechnik* (Germany), Jan., 1930, V. 8, No. 2, p. 25-31.—Old railway bridge which was badly damaged by action of water and weather was widened and provided with new reinforced concrete supports. Interesting construction methods were used for connection of old and new structure. High early strength cement was used which gave compressive strength of 330 to 380 kg./cm.^{2} after three days.—A. E. BEITLICH

Bridge over river Breitach near Riezlern (Austria). E. MUELLER. *Die Bautechnik* (Germany), Jan., 1930, V. 8, No. 3, p. 33-5.—Highway bridge over deep valley of Breitach in Austrian Alps is three hinged arch bridge of reinforced concrete. Structure consists of three parallel arches of rectangular cross-section. Span is 51 m. and its height 13 m. Roadway is supported by

eleven vertical reinforced concrete frames, six of them resting on arches, rest being placed on special foundations on both shores. Openings between frames are 5.50 m. Arches are 1.10 m. high at abutments and 0.65 m. at centers; their width is 0.70 m. Cross-sections of columns vary from 30 by 30 to 40 by 40 cm. Roadway is 5.00 m. wide with two 1.00 m. wide sidewalks on both sides. Special construction of abutments was necessary. Detailed description of reinforcements is given. High early strength cement was used for greater part of structure. Compressive strength after 6 weeks was 250 kg./cm.^{2} Foundations were made from ordinary portland cement. Scaffolding and formwork consumed much labor since high water is very frequent. Two wooden towers, 15.6 and 16.9 m. high with pile foundation supported formwork for arches. Materials were mixed in one mixer on right shore. For one cu. meter aggregates 280 kg. portland cement was used. Dump cars transported concrete to big chute.—A. E. BEITLICH

Arch bridge near Echelsbach (Germany). H. SPANGENBERG. *Der Bauingenieur* (Germany), June, 1930, V. 11, No. 23, p. 391-406.—Highway bridge over river Ammer with span of 130 m., which is longest span bridge in Germany, was opened for traffic in December 1929 after one year of construction. Bridge makes short cut and eliminates long detour of important highway. System Melan-Sachsenberg was used for construction of two halves of arch which were built without centerings. Gravel load of weight of concrete was placed upon free ends of arches to keep stresses constant. Load was decreased in same rate as construction progressed. Bridge consists of two hollow box-shaped arches of 130 m. span, placed in 6 m. distance. Ten I-beams and two U-beams at abutments connect both arches. Ribs have constant width of 1.50 m., their height increases from 2.00 m. to 3.20 m. Walls of ribs are 35 cm. thick and are stiffened by inner partitions. Abutments are 11 m. wide. Roadway, which is 8.30 m. wide, rests on 14 vertical frames of different height supported by arches. Distance between frames is 10.50 m. Their dimensions increase with their length, greatest being 1.1 m. by 0.8 m. by 24 m. Two main girders of roadway are 1.10 m. high and 0.4 m. wide with distance of 6 m. Roadslabs are 20 cm. thick and are cross reinforced. Four expansion joints were made. Detailed description of steel structure is given. Special care was necessary for closing of arches in center of bridge. Author gives discussion of reasons why box-shaped cross section of arch ribs was selected. He also gives calculations of static and dynamic pressures and stresses which occur in structure. Forms were made at site in small sections and were raised to iron structure. Two suspended platforms could be moved along inner part of arches. Aggregates were found in neighborhood and furnished very good construction material after being washed. Mixture of 40 per cent sand and 60 per cent gravel of 25 mm. size with 340 kg. cement was used for one cu. meter of concrete. Compressive strength after 28 days was 390 kg./cm.^{2} Concreting was done without any difficulties. Concrete was transported in buckets for lower parts of arch and was poured through chutes in center part. Compressed air hammers provided dense concrete. For 14 days after pouring, concrete was wetted three times per day to prevent early shrinkage. Article is accompanied by 32 illustrations.—A. E. BEITLICH

The collapse of the bridge over the river Oder near Gartz (Germany). GUSTAV HAEGERMANN. *Zement* (Germany), May, 1930, V. 19, No. 21 and 22, p. 494-7, 517-21.—Bridge was three-hinged arch which rested on two pillars in river and two abutments on land. Entire structure was made of reinforced concrete. Distance between pillars was 58.2 m. and between pillars and abutments 37.7 m. each. Height of middle arch was 10.5 m.; side arches were 7.5 m. high. Width of street was 5 m. Collapse was caused by crumbling of one of pillars. Dimensions of pillars was 11.30 m. long and 3 m. wide. Construction was nearly finished and removal of wooden planks was started, when

pillar collapsed. Middle arch fell into river and broke. One of side arches was pushed 1 m. over abutment, breaking latter, but was not damaged. This proved the good quality of reinforced concrete. Samples of pillar were taken and some of them were very soft. Calculations of chemical analysis showed that approximately 20 per cent of cement had leaked out on account of heavy motion of water. Pillars were constructed by "under-water-casting" method with concrete of 1:6 mix. Heavy steel piles were driven 15.7 m. below water level and excavation was made 12.1 m. deep. Forms consisted of 1 in. planks with 0.5 cm. wide joints. Height was 8.8 m.; foundation was 13.20 m. long and 5.5 m. wide. Concrete mixer and distributing arrangement was placed on top of foundation. During pouring of concrete an excess of water forced its way into the excavation, which prevented part of concrete from hardening and caused separation of concrete components.—A. E. BEITLICH

Winter-laid 10,000-lb. concrete on Chicago viaduct. (See PROPERTIES OF CONCRETE).

BUILDINGS

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, June, 1930, V. 36, No. 6, p. 38.—Principal feature in form work for drop panel around column head in flat-slab construction consists of square panel or platform that forms bottom of concrete drop panel. Form work for drop panel is supported in much same way as floor slab, namely, by a framework of joists and girts which in turn are supported by 3 x 4 in. or 4 x 4 in. centering posts. Two girts (one along each side of drop panel) furnish entire support for drop panel form. After panel is set in place its sides are closed with dressed 2-in. pieces whose dressed height is distance between bottom of finished concrete drop panel and finished floor slab, less $\frac{7}{8}$ in. A $\frac{7}{8}$-in. fillet is then tacked all around top of drop. Stripping of form work of the drop panel must be done in connection with stripping of rest of floor bay, as follows: (1) Assuming that "permanent" shores have been placed under surrounding floor bays and that metal forms of column capitals have been removed, loosen wedges between girts and tops of centering posts, but do not completely remove the wedges. (2) Remove two joists on one side of column by sliding them out of hangers, and let down that half of form panel. (3) Repeat foregoing operation on other side of column. (4) Remove double-headed nails from scabs. (5) Remove the 1 x 6 in. horizontal pieces that brace the centering posts, and take posts down.—C. BACHMANN

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, July, 1930, V. 37, No. 1, p. 38.—Present article deals with form work for walls above grade. They may be ordinary exterior or interior bearing walls; or they may be curtain walls extending between pilasters, supported on their own footings but carrying nothing except their own weight; or they may be supported by pilasters and have no footings. Use of 30-in. wall form units employed in foundation wall construction is not usually advisable above grade. Better workmanship is obtained if form panels are built to size of entire wall panel, extending from pilaster to pilaster and from floor to floor. Inside panel extends from completed floor or other support to underside of form boards for floor above. Height of outside panel will depend on position of belt courses or similar details. Best type of waling strip consists of two 2 x 6 in. timbers spiked together through fillers of 1 x 6 in. material, about 3 ft. apart. Space should be found to build wall form panels for first story in a lying-down position alongside place where they are to be erected. Waling strips are laid down, vertical studs tacked to them with double-headed nails, and form boards are nailed to studs. Panels are then raised from both sides, joined with column or pilaster forms and tie bolts and concrete or wooden

spreaders placed and tightened. False frames for windows and other openings are set against outside panel before inside panel is raised. Form work for wall pilasters is erected independently in manner previously described. Pilaster form work should be erected and thoroughly braced before wall form panels are raised into position to provide secure anchorage for latter.—C. BACHMANN

New Herrentor high school in Emden (Germany). HIRT. *Zement* (Germany), July, 1930, V. 19, No. 31, p. 733-4.—Building covers area of 2940 m^2 and rests on foundation consisting of 1460 wooden piles 11 m. long. Floor construction in reinforced concrete is of interest. Arrangement of reinforcements is described.—A. E. BEITLICH

Construction of building corner in reinforced concrete. THOMAS. *Zement* (Germany), June, 1930, V. 19, No. 26, p. 616-7.—Corner of building which was hindrance for traffic in narrow street, was removed and upper stories of building supported by interesting reinforced concrete construction. —A. E. BEITLICH

Concrete church Jacobi in Stettin (Germany). K. DAHL. *Zement* (Germany), July, 1930, V. 19, No. 29, p. 681-2.—Formwork, placing of reinforcements, concreting and removing of forms for church construction was carried out within 9 weeks. Six concrete frames, spaced 5.0 m. apart, support structure. Inner ceiling is made of wood in order to improve acoustics.—A. E. BEITLICH

High school construction in Kassel (Germany). WILHELM DAVID. *Zement* (Germany), June, 1930, V. 19, No. 26, p. 610-3.—Difficulties were encountered in placing foundations. Ground consisted of loose filled material to a depth of 8 m. Numerous single foundations were made on which 4-story building rests. It is 25 m. long, 15 m. wide and 9 m. high. Roof is of reinforced concrete. Several side-buildings show interesting construction.—A. E. BEITLICH

Design and construction of modern factory in reinforced concrete. (See ENGINEERING DESIGN—Buildings.)

DAMS

Water power works at river Shannon in Ireland. M. ENZWEILER. *Die Bautechnik* (Germany), June, 1930, V. 8, No. 25, p. 359-70.—Shannon river is being dammed about 16 km. upstream of Limerick near village of O'Briensbridge. Water canal of 12 km. length with capacity of 600 cu. meters of water per second connects dam and power station. Difference in water levels of 30 m. is utilized in one stem for production of 90,000 kw. with three Francis-turbines of 38,600 H. P. each. Dam is of overflow principle and has two openings of 18 m. each with rigid crests. Four openings at base are 10 m. wide. Interesting feature of dam is construction of canal for passage of fish. It is made of concrete in 16 steps, each step being 50 cm. high, at left side of dam. Dam of intake canal is of similar design. It is founded on sandstone rock and has three openings of 25 m. and one 10 m. wide passageway for ships. Total structure is 97 m. wide between shore abutments. Upper part of openings is closed by one continuous gate which extends over all three openings with total length of 90 m. It is 5.40 m. high and 2.50 m. wide and consumed 620 cu. meters of reinforced concrete for its construction. Cross section of gate is box-shaped with two horizontal inner walls. Outer walls are 27 cm., bottom and floor are 40 cm. and inner walls are 28 cm. thick. Difficulties were encountered during construction of water canal, which leads to power station. Climatic conditions were very unfavorable with heavy rainfalls. During three and a half years of construction period a total of 7 million cu. meters of

ground and one million cu. meters of rock were moved. Three reinforced concrete bridges, two of them being three-hinged arch bridges, were built over canal. Power house rests on limestone rock and is entirely built of concrete. It consists of three sections, one of which is generating room for three turbine-driven generating units; other sections are for switchboards and transformers. Station is designed for later enlargement with total of 6 turbines. Water lock at end of water canal is of interesting design. Its foundations are placed on limestone rock. For construction of this 95.50 m. long structure were used 40,000 cu. meters of concrete. It is 27 m. high. Expansion joints were placed in 30 m. distance of each other; they are sealed with copper strips which are embedded in bitumen. Lock has six openings, three of which are connected with 6 m. wide pressure pipes. Centers of water pipes are spaced 15 m. apart. Special shaped concrete walls are placed in front of openings. Pipes between water lock and power house are 44 m. long each and are supported at three places. Opening of 5 m. is in water lock as passage for water when turbines are not in use. Article is completed by 38 illustrations and graphs, showing construction in all phases.—A. E. Beitlich

Development of modern concrete fodder silos. H. Hildebrandt. *Zement* (Germany), July, 1930, V. 19, No. 30, p. 709-12.—Concrete fodder silo construction in Germany is described and advantages are discussed. Silos are air-and water-proof. Changes in temperature and internal pressure do not affect structures. Protective coatings make concrete acid resistant. Good heat insulating properties allow complete control of any fermentation process. Silage of excellent properties with high albumen contents can be prepared. Several photographs show different types of silos in use.—A. E. Beitlich

Miscellaneous

Reinforced concrete bins in the German bituminous coal region. Steinheit. *Zement* (Germany), July, 1930, V. 19, No. 27, p. 633-7.—Description of a number of coal bins under construction and completed structures. Reinforced concrete bin near Bitterfeld (Germany) with capacity of 8000 tons coal is 66 m. long, 13 m. wide and 26 m. high. Total of 13,200 cu. meters of ground were removed and 3000 cu. meters of concrete were consumed for this structure. Concrete was placed from distributing tower at rate of about 200 cu. meter per day. Compressive strength after 14 days was between 140 and 160 kg./cm.^{2} Coal distribution tables and conveyor belts are underneath bin. Foundation of coal bin near Meuselwitz consists of continuous concrete slab. Capacity is 1000 tons coal; building is 63 m. long and 7.40 m. wide. Several other structures are described.—A. E. Beitlich

Ground construction. "Der Grundbau." L. Brennecke and Erich Lohmeyer. 1930, 4th edition, Wilhelm Ernst und Sohn, Berlin, R. M. 25.00. Reviewed in *Zement* (Germany), July, 1930, V. 19, No. 28, p. 666.—Book deals with all construction methods for foundation jobs. Special consideration is given to reinforced concrete piles and breakwater construction. Second volume which is still in preparation will complete work on this subject. —A. E. Beitlich

Concrete of any specified proportions produced by Rochester, N. Y., plant. *Pit and Quarry*, May 7, 1930, V. 20, No. 3, p. 76-78.—Describes ready-mix plant of Whitmore, Rauber & Vicinus, near center of city, which delivers to any job in city within 25 min. Firm has been handling aggregates for years. Aggregate portion of plant has capacity of 25 cars per day. Sand and gravel, received in bottom-dump railroad cars, are discharged into track hopper from

which belt feeder loads to inclined belt conveyors. These discharge the material through swivel chute to nine bins or to another belt conveyor which leads to mixing department. Cement is purchased in bulk and unloaded from box-cars by hand-scraper into hopper which feeds chain-bucket elevator. Cement is delivered to two of six bins above mixing department, each of 250-bbl. capacity. Sand and gravel leave bins through quadrant gates, cement through sliding gates. Weighing hopper is equipped with 3-beam scale having dial that registers only last 400 lb. Weighing hopper discharges to another hopper to await dumping into mixer at proper moment. Thus three batches are always in process simultaneously. Water is measured accurately by means of syphon arrangement. Mixer is of 2½-cu. yd. capacity and prepares an average of 350 cu. yd. in 9-hr. day, although it has prepared 400 cu. yd. Mixer is driven by 40 H. P. motor. One man controls mixing department. Concrete is delivered in trucks with bath-tub bodies. Four illustrations.—A. J. HOSKIN

Placing concrete by compressed air. C. H. MUNRO. *Australasian Engineer*, June 7, 1930, V. 30, No. 168, p. 5.—One of earliest pneumatic placers worked on the following principle, and modern methods are but improvements on it: A mass of concrete is placed in a chamber with restricted outlet, and uniform air pressure applied towards the outlet. When, and not until, a further jet of air is blown into the mass near outlet, a small portion, or sub-mass becomes detached and forced into delivery pipe. As each submass is forced into the pipe, the pressure falls and pressure behind mass forces more up to outlet. Process is thus repeated continuously. In later refinements rise in pressure behind the main mass due to cartridge effect of the construction, operates a piston valve in secondary jet line when it reaches a given pressure, adjusted by springs. In practice both valves open almost simultaneously. Various works, mostly tunnels, on which this method of placing concrete has been successfully employed are described. Generally the guns do their own mixing.—JOHN E. ADAMS

Vibrated concrete. E. TREVES. *Le Genie Civil*, (France) June 28, 1930, No. 2498, p. 636.—Vibrated concrete was first tested by Freyssinet in 1917. Making use of compressed air he has since successfully used this method on large scale reinforced concrete construction. Surface vibration was patented in France by M. Berenguier and was commercialized by the society "La Route" since 1927. The vibration of the mass was first introducted by Deniau and has since 1928 been commercially exploited by the Societe des Procedes Techniques de Construction. Advantages of vibration are: perfect homogeniety of concrete, even for dry consistencies; partial elimination of air and water; constant quality concrete permitting a reduction of factor of safety; early removal of forms; good agreement of compressive strengths and their increase from 30 to 200 per cent; constant flexural strength, 16 per cent greater than that of surface vibration only; increased density (1 cu. m. of concrete requires tamping 1200 liters of material; vibration requires 1300 to 1400 liters and pervibration (vibration of the mass) requires 1450 liters. The volume of voids is reduced by one half. The rising of laitance, due to vibration secures a smooth surface. Porosity is reduced and thus danger of infiltration and corrosion by noxious agents. Vibrated concrete is easily adaptable to decorative effects.—M. A. CORBIN

Concrete and reinforced concrete construction. "Fertigkonstruktionen im Beton- und Eisenbetonbau." A. KLEINLOGEL. 1929, Wilhelm Ernst und Sohn, Berlin, 91 pp. R. M. 8.60. Reviewed in *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), May, 1930, V. 82, No. 21-22, p. 187.—Advantages are described of structural parts of concrete and reinforced concrete which are made in shop ready to put together on job. No forms are necessary and construction can be carried out at much greater

speed. It can be applied to different types of floor construction, large halls, vaults, bridges, breakwaters and railroad structures.—A. E. BEITLICH

Blame divided for Detroit sewer failure. *Eng. News Record*, Aug. 7, 1930, V. 105, No. 6, p. 210-211.—Insufficient soil investigation, inadequate design, inspection and supervision, together with faulty construction, were held by investigating committee to be contributing causes to failure of Detroit sewer. Responsibility was placed jointly on department of public works and contractor. Failure was caused directly by removal of foundation composed of fine water-bearing sand which was carried into the interior of the barrel by groundwater through faulty concrete construction joints and openings developing in barrel after construction. Physical inspection of concrete showed proper proportioning but faulty workmanship in placing, resulting in porous low-strength concrete and thin walls. Thickness of sewer walls was found to vary from 6¾ in. to 26 in. Specifications required minimum thickness of 16 in. Tests of cylinders cored from sewer barrel showed compressive strengths varying from 987 to 6,171 lb. per sq. in. Samples of concrete analyzed in the laboratory showed proportions ranging from 1:2.6:1.8 to 1:3.4:5.3. Although these tests showed evidence of too much sand in the concrete, it might have been due to finely divided quicksand being carried into concrete by infiltering ground water and is not necessarily proof of an oversanded original mix.—D. E. LARSON

Foundations of gas engine plant in Hamborn (Germany). *Zement* (Germany), June, 1930, V. 19, No. 23, p. 541-3.—Foundations for power house with four gas engines is described. Engines are driven with blast furnace gas and furnish compressed air for blast furnaces. Strong vibrations of explosions require special foundation construction. Detailed description of reinforcement is given. Total of 10,500 cu. meters of concrete with 1350 tons of steel were used for foundations; approximately 130 kg. steel were placed in one cu. meter concrete. Water cooling plant and other minor structures consumed 5000 cu. meters concrete.—A. E. BEITLICH

Modern elevator for construction materials. *Zement* (Germany), June, 1930, V. 19, No. 25, p. 595-6.—New type of elevator for rapid transportation of structural materials is described. Highest permissible load is 600 kg. Elevator is easy to install and to move. No scaffolds are necessary.—A. E. BEITLICH

Concrete and reinforced concrete in electro power station construction. BREITUNG. *Zement* (Germany), June, 1930, V. 19, No. 25-26, p. 586-91, 613-6.—Article describes numerous possibilities for application of concrete and reinforced concrete in construction of water and steam power plants for production of electrical energy. Concrete is favorable material for dams and hydraulic structures since good combinations of cements and aggregates guarantee high strength and impermeability. Pressure pipes and intakes can be made of reinforced concrete. Foundations for water and steam turbines can be designed to reduce vibrations to minimum. In generating plants which use steam, concrete is used for construction of coal bins, water tanks, steam condensers and cooling towers. Large field for use of concrete is construction of stacks and poles. Nineteen photographs and drawings are reproduced.—A. E. BEITLICH

Under-water-casting of concrete. W. NAKONZ. *Die Bautechnik* (Germany), Jan., 1930, V. 8, No. 3, p. 35-6.—Methods of casting concrete foundations under water are described. Directions are given for correct working of concrete. Casting must be carried out without any interruption. Funnel must be always filled and movable in all directions. Excavation must be pro-

tected from running water which might cause excessive loss of cement. Difficulties increase with increasing depth of foundation. More than one funnel has to be used when large foundations are placed. Concrete should be earth-moist in order to prevent too rapid flow of mixture. All advantages of different methods and several possibilities for their application are described. —A. E. BEITLICH

Ready-mixed concrete plant for small city. *Concrete*, July, 1930, V. 37, No. 1, p. 27.—Ready-mixed concrete requirements of small city are well covered by small plant of W. A. Gingrich, in South State Street, Ann Arbor, Mich. Sand and gravel are brought, in proper proportions, from commercial pit three miles away. Only material stored at plant is cement. One man employed brings mixed sand and gravel from pit, dumps it into skip of mixer, brings cement from warehouse, charges mixer and sets it into operation, then drives truck around to other side of mixer, dumps concrete and delivers it. Mixing proportions, as a rule, are 1:2:4, but customers are supplied with any proportions desired. A water-control device regulates quantity of mixing water and maintains uniformity from batch to batch. Price for this mixture is $6.50 per cu. yd., delivered.—C. BACHMANN

Chemical technology for contractors. "Chemische Technologie fuer Bauingenieure." DR. FRANZ HEMMELMAYER. 1930, Ferdinand Enke, Stuttgart (Germany). 123 pp., 27 illustrations. R. M. 8.50. Reviewed in *Zement* (Germany), June, 1930, V. 19, No. 23, p. 548.—Book is one volume in Enke's "Collection for Chemistry and Technology." It describes all chemical reactions and materials which are of interest to the contractor. Four parts deal with following subjects: Water and water purification, illumination, explosives and structural materials.—A. E. BEITLICH

New forms and simple apparatus to determine the water permeability of concrete. ERNST RISSEL. *Zement* (Germany), June, 1930, V. 19, No. 23, p. 532-5.—Old methods of determining water permeability of concrete show several disadvantages. Cross-section through which water penetrates cannot be exactly determined. Direction which is studied is mostly vertical to actual direction of water penetration in practice. New forms are described, which allow examination of both directions. Concrete joints are also taken into consideration. Sawing of concrete test cylinders is not necessary and forms can be made with very little costs. Exact dimensions and directions for the preparation of the test pieces are given. Pressure is kept constant by 13 m. high water column. Five test pieces can be connected to this water line. Temperature in room is kept between 15° and 20° C.—A. E. BEITLICH

Reinforced concrete storage silo for malt. F. KOENIG. *Zement* (Germany), July, 1930, V. 19, No. 31, p. 735-6.—Silo which consists of 16 separate compartments has capacity of 4900 cu. meters. Underground was very soft and structure was placed on strongly reinforced Pilz-slab. Twenty-five concrete columns distribute load on foundation. Walls were constructed with sliding forms; they are 17.10 m. high and 15 cm. thick. One cu. meter concrete was made with 310 kg. cement. Cube tests showed 200 kg./cm.^{2} strength after 28 days.—A. E. BEITLICH

Use and abuse of building materials. A. M. GILES. *Builder* (England) June 20 and June 27, 1930, V. 138, Nos. 4559 and 4560, p. 1192 and 1235.—Concrete: It is essential that dry ingredients be well mixed before water is added or voids will not be filled, and sufficient water must be added to cause complete hydration. River water is generally satisfactory if uncontaminated by factory refuse. Sea water may sometimes be used where setting time and appearance are not important. Contaminated water should not be used unless 28-day compression tests give results not less than 80 per cent of clean

water values. It is suggested that correct amount of water may be taken as 28 per cent of weight of cement plus 4 per cent of weight of combined aggregates in mix. The slump test is given in detail, and its frequent application recommended. In experiments with 1:2:4 mix, 17 per cent shrinkage in volume was observed after mixing dry. Final decrease after 28 days, was 27 per cent volume of dry materials. Methods of hand and machine mixing are discussed. For machine mixing, dry materials should be mixed 15-20 secs. Tests show that 5 min. mixing gave 3200 lb. per sq. in. concrete in 28 days, and 1 min. mixing 2900 lb. per sq. in. Many faults are due to insufficient curing. Concrete should be kept damp for two or three weeks. Forms left in place as long as possible retard evaporation. Setting time is determined by flow table. All concretes are more or less permeable, and circumstances affecting this property are many. Clean sand and correct amount of water are main essentials.—JOHN E. ADAMS

ROADS AND PAVEMENTS

Steel forms for road work. *Indian Concrete J.* (India), June 15, 1930, V. 4, No. 6, p. 172.—A new type of steel form is illustrated, which may be used for any thickness of slab between $3\frac{1}{16}$ in. and 9 in. It consists of two steel channels of different sections and a central steel flat. The latter is $1\frac{1}{2}$ in. narrower than the space between the channels and is connected to the support by a countersunk bolt and nut moving in a slot. This arrangement permits vertical movement and leaves spaces through which the reinforcement may pass. No fixing or pegging down of forms is required and there are no loose parts. The method is also applicable to curbing.—JOHN E. ADAMS

Examples of road construction in Vauclose. *Australasian Engineer*, June 7, 1930, V. 30, No. 168, p. 4.)—Road construction in Vauclose is mainly confined to the concrete and bituminous macadam penetration types. The latter is more favored since reasonable serviceable roads are preferred to those with no maintenance costs, but higher first costs. A macadam surface, practically free from waviness has been obtained by handpacking foundations in squares, to a depth of 9 in., the grain of the stone being laid across the road to avoid splitting. For concrete roads longitudinal screeding is recommended because despite the greater number of joints the grain of the surface runs with the traffic.—JOHN E. ADAMS

SHOP MANUFACTURE

Concrete products industry needs more substantial equipment. *Concrete*, August, 1930, V. 37, No. 2, p. 41.—There is little question that the products industry is due for a rather thorough overhauling in matter of equipment, even though several years may elapse before the process is completed. The industry has come a long distance since the day when concrete block machines exhibited at annual cement shows consisted of light-framed contraptions operated by a foot pedal. Now industry is installing heavier and sturdier equipment, fully automatic in its operation. Advantages of this modern equipment are many. It is labor-saving; it places a plant on a large production basis, for a single machine capable of producing six or eight units of standard size in a minute will have a capacity of 3,600 or 4,800 units in 10 hours of actual operation.—C. BACHMANN

Preparing concrete masonry advertising. T. A. DAY. *Concrete*, July, 1930, V. 37, No. 1, p. 35.—Probably best medium for local manufacturer is the newspaper reaching prospective home owners in his community, if he intends to spend his advertising money in residential field. Choose the newspaper that will carry the message to the greatest number of prospects. It is

generally agreed that manufacturer can make no better investment in his merchandising program than to tie in his newspaper advertising with educational copy provided by Architects' Small House Service Bureau. By placing his advertisement in issues in which concrete masonry house is described, he obtains three or four times the advertising space devoted to concrete masonry that he would ordinarily obtain. Success in advertising depends greatly upon ability of advertiser to make reader remember him and his product. Advertisers rely upon repetition of outstanding points to help them obtain new acquaintances and additional users.—C. BACHMANN

Successful piece-work pay system in concrete products plant. R. J. FISHER. *Concrete*, June, 1930, V. 36, No. 6, p. 16.—In general, there are three piece-work payment plans. One consists of paying foreman for all labor. Second system consists of definite apportioning of pay for labor among all men. Third system consists of paying mixer operator, machine operator and off-bearers on piece work basis and hiring day labor for curing room and stock piles. The Economy Concrete Products Co., Milwaukee, Wis., has adopted system under third classification. Prices for each unit of work were determined and set of rules and regulations adopted. These were presented to employees in a letter. Under old system, production per man per 10-hr. day for 8, 10, and 12 in. units averaged about 750, 725 and 500 units respectively. New system averages about 1100, 1050, and 675, respectively. This increase was brought about by using two men to the machine instead of usual three. Production for each machine is but slightly less, from about 2½ per cent for the 8 in. unit to about 10 per cent for the 12 in. unit. The price of 60 to 95 cents a hundred for units from the 4 in. to the 12 in. size is enough incentive to bring production close to the maximum. System is too new to get a detailed report on costs, but taking the average of the 6, 8, 10 and 12 in. units, the cost has been reduced by 0.4 cents per block. It was formerly 2 cents. Under old system it was not uncommon for production to drop 20 to 25 per cent below average daily production, causing wide variation in the cost per unit. Under the new system, labor cost per unit remains the same. Saving is also made in unloading cement. Price of ¾ cents per bag looks low, but has netted men an average of 83 cents an hour. A thousand-bag car is unloaded by nine men in less than an hour.—C. BACHMANN

Well-designed, attractive products plant is credit to industry. *Concrete*, July, 1930, V. 37, No. 1, p. 20.—Cement Products Co., Davenport, Iowa, has a large volume of business in its cast stone work. This means that among the visitors to the plant are people of influence. In their visits they are often investigating merits of material made at this plant as compared with natural stone, or with cast stone made elsewhere. Plant has real business office, located on second floor of building. Principal office space is divided into front office and two private offices. Farther on is the office of the sales manager, while at the far end in the corner of the building, is the office of the president. Building is 2-story structure, built of high-grade concrete units. The structure is 138 ft. by 51 ft. The offices and drafting rooms on second floor occupy a strip about 14 ft. wide. Manufacturing space is rather symmetrical in plan, about 138 ft. long and 38 ft. wide. Each end of the manufacturing space is occupied on ground level by curing kilns, four in each end, and each kiln is equipped with two tracks. Above curing kilns is reinforced concrete floor slab which provides a floor space of about 34 by 40 ft. at each end of the building, these two floor areas being joined by a balcony floor.—C. BACHMANN

Materials

Admixtures

American experiments and experiences with Tricosal. A. Kleinlogel. *Zement* (Germany), Aug., 1930, V. 19, No. 32, p. 758-60.—Properties of Tricosal were carefully examined and it was found that water addition for making of concrete can be lowered about 10 per cent when 1 per cent Tricosal is added. Compressive strength was increased 18 per cent with 1 per cent Tricosal while tensile strength was raised about 43 per cent. Additions of Tricosal improve impermeability of concrete. Briquets were broken and cemented together with Tricosal. After 48 hour storage in moist closet briquets were broken again and all showed new breaks at different places.—A. E. Beitlich

Waterproofing of concrete structures. (See Field Construction—Miscellaneous.)

Lactic acid, mortars and brick floors. (See Properties of Concrete.)

Aggregates

New sand and gravel operation in Philadelphia territory. *Rock Products*, May 10, 1930, V. 33, No. 10, p. 49-53.—Log washers and special types of screens are used to make concrete aggregate from bank material that contains considerable quantities of clay which had hitherto been found difficult to remove. Operation of machines is described in detail with flow sheets and quantities.—Edmund Shaw

Brick and stucco mortars. K. A. Goslich. *Tonind. Ztg.* (Germany), 1930, V. 54, p. 1079-83.—Author thinks that the sand should have sharp corners. After reviewing literature on effect of clay and loam, he points out value of clay in small amounts in filling voids, but states organic matter of humic type must be absent. For stucco the sand must be finer than for brick laying. Also for stucco it has been found (in Germany) that sand containing original moisture makes stronger mortar than if it is allowed to dry out.—F. O. Anderegg

Selection and acceptance of sand and gravel for concrete, especially reinforced concrete. Otto Graf. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 71, p. 1159-60; No. 72, p. 1175-7; No. 74, p. 1203-5.—After the specifications of concrete are determined, careful selection of materials is needed to meet the specifications safely without running too far over. Quality of cement should be uniform. Aggregate should be fairly uniform in grading and to secure this result, sand and gravel should be separated by a ¼-in. screen (7 mm. holes). Gravel should not run much larger than 1¼ in. (30 mm. holes) and should be nearly free from flats. Effect of grading of sand on density, strength resistance to corrosion, etc., of the concrete has been studied and best results are obtained with Fuller curve grading, except that a little more fine material increases resistance to corrosion. Grading recommended is consistent with German and American specifications and is approached by many commercial aggregates.—F. O. Anderegg

Tentative specifications for light weight cast stone (Schwemmsteine) from Bims aggregate. German Standards Committee. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 69, p. 1133.—Bims aggregate is a porous pumice stone which is cast into blocks with lime, standard cement or special cements as binder. The standard shape is 4 x 5 x 10 in., but others are allowed. The maximum weight for this size is 5.34 lb. For compressive strength the flat sides are made parallel with mortar and the air-dry strengths on 10 stones must average 170 lb. per sq. in. when lime is used as the binder, 286

with standard cements and 419 with special cements. The stones must withstand 25 cycles of freezing to 25°F. when saturated with water followed by thawing. These stones have been recognized as fire resistant by the Prussian Ministry for the Peoples Welfare (Voklswohlfahrt). The sp. heat conductivity should range from 2.5 to 3.1 B. T. U. per hr., per sq. ft., per in., per deg. F. difference.—F. O. ANDEREGG

Effect of addition of finer sizes to gravel on strength of concrete. STANTON WALKER. *National Sand Gravel Bul.*, August, 1930, V. 11, No. 8, p. 5-11.—An investigation of strength of concrete, coarse aggregate for which contained varying amounts of finer sizes of gravel, has been carried out in the research laboratory of the National Sand and Gravel Association. This report gives detailed results of tests and some studies of them. Gravel deposits, because of method by which they were formed, often contain an excess of certain sizes which are limited by specifications based on use of arbitrary proportions. Common size to occur in excessive amounts is that generally known to industry as "pea" gravel. Elimination of these sizes often constitutes an economic waste which could be avoided by proper design of the concrete. Ten different gradings of gravel were used, the differences being in amount of No. 8 to No. 4 and No. 4 to ⅜ in. sizes added to a ⅜ to 1½ in. gravel. The tests were divided into two groups. First consisted of studies of 10 gravel gradings in four concrete mixtures as follows: 1-1½-4½, 1-2-4, 1-2½-3½ and 1-3-3. In second group one grading of gravel was tested in mixtures ranging from 1-1-2 to 1-3-6, each containing sand and gravel in proportions of 1 to 2. Compression tests of 6 by 12-in. concrete cylinders and transverse tests of 6 by 6 by 30-in. beams were made. In the concluding remarks the following statements are made: (1) For standard method of molding specimens used in this investigation, it is apparent that 1-1½-4½ mix contained insufficient sand to provide proper workability. (2) Reductions in strength on account of inclusion in ⅜-in. to 1½-in. gravel of relatively large quantities of material between ⅜-in. and No. 8 sieve were comparatively small. Inclusion of 30 per cent of No. 4 to ⅜ in. size decreased strength less than 10 per cent when comparisons were made for same arbitrary proportions. If quantity of sand in mixture is reduced to compensate for increased amount of finer material in the gravel and differences in cement content taken into account, strength is increased in some cases and in other cases reduction is small. (3) In general, reduction in strength on account of the addition of finer sizes was in proportion to additional quantity of mixing water, expressed as a ratio to volume of cement, required to maintain same workability. (4) Both compression and transverse tests in this investigation showed definite relationship between strength and quantity of mixing water, expressed as ratio to volume of cement, and indicated that mixing water was most important factor in controlling strength of concrete. (5) These data indicate that considerable economies may be effected by utilization of finer sizes of gravel.—C. E. PROUDLEY

Aggregate for Boulder Dam. (See FIELD CONSTRUCTION—Dams.)

CEMENT

Determination of sucrose soluble CaO in portland cement. T. F. SEDGWICK. *Rock Products*, May 24, 1930, V. 33, No. 11, p. 81.—Author, an industrial chemist of Honolulu, T. H., states he has found that method of determining sucrose soluble CaO in burnt and hydrated lime appears to give a fair index of the free lime in portland cement. He quotes standard methods for finding sucrose soluble CaO.—EDMUND SHAW

Wage rates and labor conditions in the cement industry in Europe. FRANCIS MILTOUN. *Pit and Quarry*, April 23, 1930, V. 20, No. 2, p. 37-55.—

Economic discussion, by an American who resides in Europe, of the protection given by European countries to their respective cement industries. Wage scales, productions and exports are covered for Sweden, Germany, Belgium, Denmark, France and Italy.—A. J. HOSKIN

The cement industry in Bologna. ANGELO LILI. Reprint from *Il Comune Di Bologna* (Italy), March, 1930, 4 p.—A general description of investigations made in Bologna, which have resulted in conclusion that it is logical location for cement mill. Previously unknown deposits of pure limestone were located and tests show that either natural or artificial cements can be economically produced. National legislation to stimulate construction and thus provide an outlet for production of cement mills in the country is recommended.—C. G. CLAIR AND M. N. CLAIR

A new theory for the fine grinding of cement. O. RAULIN. *Ciment* (France), 1930, V. 35, No. 9, p. 342. Cement may be ground too fine to give best results, the explanation given, being that the air cushion on very tiny particles prevents them from being hydrated. Each cement has a critical grain size which depends upon the burning and which gives the best reaction with water. This critical size should be determined for each cement and then adhered to.—F. O. ANDEREGG

Improvements in cement mill practice. A. B. HELBIG. *Zement* (Germany), Aug. 21, 1930, V. 19, No. 34, p. 796-9.—Author emphasizes importance of installation of air separator in cement grinding mill. Different types of separators are characterized, their advantages described and calculations of their power consumption given. (cf. A. B. Helbig, *Zement* (Germany), Jan., 1930, V. 19, No. 2, p. 25-8 and March, 1930, V. 19, No. 11, p. 237-9.)—A. E. BEITLICH

Practical guide for chemists in the cement industry. "Guide practique du chimiste dans l'industrie du ciment." C. TSOUNTAS. 1929, Editor: *Revue des Materiaux de Construction*, Paris (France), Fr. 20.00. Reviewed in *Zement* (Germany), Aug., 1930, V. 19, No. 32, p. 760.—Book describes analysis of cement, raw materials, fuels, gases, lubricants and water. No consideration is given to physical examination of finished cement. Laboratory test is described which consists of determinations of insoluble residue of ignited raw mix after certain time intervals. It is claimed that results obtained throw light on behavior of clinker in kiln.—A. E. BEITLICH

Burning of lime and cement. J. DEFORGE. *Rev. Materiaux Construction Trav. Publics* (France), 1930, No. 249, p. 212-6; No. 250, p. 262-6; No. 251, p. 298-303.—In the vertical kiln one difficulty is to get the gases properly distributed. One way to improve this is to introduce part of air admitted for combustion through a central chimney. Another kiln is described where producer gas used for fuel is admitted about ⅓ of the way up. Combustion air comes up through the hot lime. The top is closed and the gases are pumped out, being about 40 per cent CO_2 with very little CO, as compared with most shaft kilns where usually relatively large amounts of CO mean waste heat. The CO_2 is purified, cooled and condensed to a liquid; or it may be used in precipitating chalk or for other similar purposes.—F. O. ANDEREGG

Slurry filtration in manufacture of cement. GUSTAV BOEHM AND DESIDER STEINER. *Zement* (Germany), Aug., 1930, V. 19, No. 33, p. 768-75.—Directions for installation and operation of slurry filters in wet process cement plants are discussed. Heat calculations illustrating economy of process are given and tables show relations between water content and specific gravity of slurry. Preliminary tests must be carried out to find suitability of slurry for

filtering process. Slurry which tends to sedimentation is usually well suited for filtration while that which stays in suspension very long is hard to filter. Great uniformity of lime content as well as fineness of slurry is necessary. High temperatures have favorable effect; during winter slurry should be preheated and special care taken to prevent blocking of filters and slurry pipe lines. Continuous stirrers and cleaning every two weeks are suggested. Several different kinds of filters are discussed and their power consumption given. Engineering tests which were made to compare economy of wet process with filter method showed great superiority of the latter.—A. E. BEITLICH

Concrete of great soundness made up of portland cement and arsenious oxide. DONOVAN WERNER AND STIG GIERTZ. *Teknish Tidskrift* (Sweden), April, 1930, No. 5, p. 41-49.—The Royal Engineering Academy of Sweden has for one year carried on tests with a mixture of portland cement and As_2O_3, to find some use for As_2O_3 obtained as a by-product in production of iron in Sweden. This kind of cement is recommended for water resistant concrete, sewer concrete, etc. It was found that setting time could be controlled by heating the mixture. Best result was obtained with a mixture of 70 per cent cement and 30 per cent As_2O_3. For temperatures between 320° and 374° F., setting time was 1 to 3 hours. Higher temperatures delayed the setting. Strength was found greatest for the 30 per cent mixture. Compressive strength of mixture at 7 days was 3890 lb. per sq. in. compared with 2720 lb. for neat cement. After 3 months the corresponding values for the mixtures were found to be 5760 and 7920 lb. per sq. in., respectively. The soundness of the 30 per cent mixture proved to be very good. This mixture decreased the dissolving of CaO from .0507 for 100 per cent cement to .0045. Complete article includes 32 graphical tables.—O. ALBERT.

Is the regulation of the rotary kiln for uniform high efficiency made difficult by great variations of the silica modulus? OTTO FREY. *Zement* (Germany), Aug. 7, 1930, V. 19, No. 32, p. 744-5.—Experiments were carried out with raw materials, which showed great variations in silica content (silica modulus ranging from 2.2 to 7.0). Raw mixture was so proportioned, that modulus was between 3.0 and 3.5. No unfavorable effect during burning process could be noticed. Following observations were made: No ring formations took place when silica modulus was between 3.0 and 3.5; when modulus sank below 3.0, a ring was formed; clinker was black in color and very hard; cement produced was of high quality; sintering point of raw mixture was 1520° to 1530° C. and fuel consumption was 25-26 per cent. Kiln lining was made of Dynamidon bricks.—A. E. BEITLICH.

Breaking down of tricalcium silicate by heat. S. L. MEYERS. *Rock Products*, April 12, 1930, V. 33, No. 8, p. 78-79.—Author found increase of free lime after ignition of cement, the increase being greater with higher temperature and with longer time of heating. This is ascribed to breaking down of tricalcium silicate to dicalcium silicate by heat with consequent liberation of free lime. Experiments on 4 $CaO:Al_2O_3$, Fe_2O_3, 3 $CaO:Al_2O_3$, 2 $CaO:SiO_2$, and 3 $CaO:SiO_2$, all prepared with pure materials, showed that only 3 $CaO:SiO_2$ liberated free lime on ignition. In heating ordinary portland cements, as much as 11.28% of lime has been liberated corresponding to the breaking down of 45.98% of tricalcium silicate to dicalcium silicate. A slightly higher liberation of free lime was obtained with high early strength cement. Clinker or unground cement did not show this reversion with heat. Aged cement was found more susceptible to heat reversion than fresh cement.—EDMUND SHAW

Disturbing effects in portland cement and high alumina cement mixtures. HANS KUHL AND SETSUO IDETA. *Zement* (Germany), Aug. 21,

1930, V. 19, No. 34, p. 792-5.—Portland cement clinker was ground together with different amounts of calcium—alumina melts of different compositions, prepared by fusing mixtures of $CaCO_3$ and Al_2O_3 in oxy-acetylene flame. Composition of melts varied from 3:5 to 4:1 ($CaO:Al_2O_3$). Portland cement clinker and alumina melts were ground separately to fineness of 10 per cent residue on 175 mesh sieve and mixed in proper proportions with an addition of 3 per cent gypsum of same fineness. Following mixtures were prepared: portland cement: alumina melt, 90:10, 75:25, 60:40, 40:60, 25:75 and 10:90. Tensile strength after 1, 3 and 7 days water storage was determined by small-piece testing method proposed by Kuhl. Authors drew following conclusions: melts similar to high alumina cement decrease strength of portland cement after one day when 25 to 40 per cent is present. Alumina melts of composition $2CaO:2.5\ Al_2O_3$ have disturbing effect only when 60 to 75 per cent is mixed with portland cement. When ratio $CaO:Al_2O_3$ becomes greater than 2.5:1, melt has tendency to increase portland cement strength slightly. It is stated that reactions take place between calciumhydroxide, which is formed by hydration of portland cement, and aluminum hydroxide from alumina melt.—A. E. BEITLICH

Experience with slurry filters at the Osborn, Ohio, plant of the Southwestern Portland Cement Co. EARL C. HARSH. *Rock Products*, May 24, 1930, V. 33, No. 11, p. 51-54.—Filters are of the American continuous type. Improvements noted from their installation are: increased kiln capacity, decreased coal consumption, material increase in steam generated by waste heat boilers. Formation of mud rings in kilns has been obviated and clinkers are smaller and burning more uniform. Only one of two 9-disk filters installed has been found necessary. Each 12-ft. disk of 15 sectors has 200 sq. ft. filter area. Palm twill, 15 oz. fabric has given best results. Life was 25 days but by treatment has been increased to 40 days. Cycle uses 40 per cent of time for pickup, 40 per cent drying and 20 per cent discharging. A 28-in. vacuum is maintained by duplex pump and motor. 3 h. p. variable speed motor drives filter through specially designed speed reducer at one revolution in seven minutes. The filter cake goes to a pug mill and is mixed with dust collected by Cottrell precipitation plant. Raw material is ground to 92 per cent passing 200 mesh. Slurry has 35 per cent water which is reduced to 19 per cent by filtering, a removal of 180 lbs. water for each bbl. of cement made.—EDMUND SHAW

Studies on lime—alumina cement I. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi* (*J. Soc. Chem. Ind.* (Japan), May, 1930, V. 33, Supplemental binding, No. 5, p. 167-9.—Raw materials which are suitable for manufacture of a lime—alumina cement in Japan were investigated. This kind of cement was proposed by Richard Grün (Tonindustrie Zeitung (Germany), 1924, V. 48, p. 247) and contains approximately 15 per cent silica, 50 per cent lime and 28.2 per cent alumina and ferric oxide. Following raw materials can be used: limestone, puzzolana, loam, colloidal earth and diaspore. Analysis and compositions of these materials are given. Raw mixtures were sintered or melted in electric resistance furnace at temperatures ranging from 1300° C. to 1380° C. Tables show chemical composition of these burns. Hydraulic properties of products obtained were studied by testing on compressive strength of 1:3 cement—sand mortar with small-piece testing method. Cements lose strength in water curing at 7 days with a further considerable decrease at 28 days, but show great increase in strength at 28 days combined curing. It is stated that $5CaO.3Al_2O_3$ or $CaO.Al_2O_3$ are main components in lime—alumina cement from which hydrated product $3CaO.Al_2O_3.3CasO_4.nH_2O$ is dissolved in water which fact decreases its strength. Further studies will cover dusting of heated products, decrease of strength in water curing and increase of strength in air hardening.—A. E. BEITLICH

Long aerial tramway carries limestone to cement mill in the Cascades. FRANCIS CHURCH LINCOLN. *Pit and Quarry*, April 23, 1930, V. 20, No. 2, p. 41-48.—This is a complete description of Superior Portland Cement Company's plant and quarry near Concrete, Wash. Company succeeded Washington Portland Cement Co., the oldest cement company in the state. Limestone occuring in steeply-dipping beds, of various colors and qualities is raw material for wet process. Recrushed stone is belt-conveyed to the loading bin of a double-rope aerial tramway, whence it is transported 6,671 ft. to cement plant at rate of 260 tons per hr. Clay is taken from pit in 5-ton trucks to storage pile at plant. Slurry is pumped to an agitator trough and distributed to tube mills. Six kilns have a combined capacity of 5,200 bbl. per day. Fuel is pulverized coal.—A. J. HOSKIN

Iron oxide vs. alumina as a fluxing agent in the manufacture of portland cement. KATSUZO KOYANAGI. *Rock Products*, May 10, 1930, V. 33, No. 10, p. 82-83.—Author comments on the work of Anton Blank (c. f. *Rock Products*, May, 1928,) with iron oxide and alumina as fluxes in cement. He reviews his own experiments, given in detail, which show that with cement made by Cichibu Cement Co., Tokio, Japan, increasing the Fe_2O_3 content does lessen coal consumption and give a higher tensile strength as Blank claims but it also gives a lower compressive strength. With very high Fe_2O_3 content both tensile and compressive strengths decrease and cement becomes unsound. He notes that Blank experimented with high alumina cement while he experimented with low alumina cement. He concludes that when cement is high in alumina and low in silica, iron oxide can be advantageously used as a fluxing agent, but when cement is low in alumina and high in silica it is better to increase the alumina content a little, leaving the iron oxide content constant. Many tables show results from which conclusions were drawn.—EDMUND SHAW

Analysis of waste gases of rotary cement kilns and their part in kiln control. ALTON J. BLANK. *Rock Products*, May 10, 1930, V. 33, No. 10, p. 58-60.—Comparative tests on the Akron and Orstat apparatus showed they agreed on the CO_2 content of waste kiln gases within 0.1 to 0.2 per cent. Oxygen contents by the Akron device were somewhat erroneous. Carbon monoxide in waste gases was shown by the blackening of silver foil placed under discharge water pipe of Akron device and kiln operators noted this in controlling kiln operation. The reasons for changes in the CO_2 record, such as variation in amount of feed and fuel and the CO_2 content of materials and air and moisture admitted, are listed. Theoretical tables and charts were prepared showing fuel oil consumption for varying percentages of free oxygen in kiln gases and for CO_2 content of kiln gases. These were checked with actual measurements of fuel oil burned and found to be more or less in agreement. Author shows how examination of record may avoid fuel losses.—EDMUND SHAW

Progress in cement research during year 1929. C. R. PLATZMANN. *Zement* (Germany), July 31 and September 4, 1930, V. 19, No. 31 and 36, p. 722-4, 843-7.—Author gives comprehensive bibliography on subjects of recent cement research. Considerable work was done to find favorable relation between chemical constituents, which were expressed by certain moduli, and formulas to obtain best possible cement composition. In close connection with this work were experiments to represent cement constitution and composition by means of graphs and curves. Reactions in kiln and rate of formation of calcium silicates and other clinker minerals were studied by several investigators. Mechanism of setting and hardening process and role of manganese in portland cement clinker were research problems. Synthesis and hydration of calcium aluminates, as well as conditions under which these aluminates form

and exist, were carefully studied. Effect of water soluble admixtures on setting and hardening process of portland cement and high alumina cement was examined. A number of articles deal with relation between fineness of cement and its strength and setting properties. Numerous new methods for chemical analysis and determination of particle size of cement were published. A small-piece testing method, which was developed by Hans Kuhl and modified by S. Nagai, was applied to scientific cement research. Change from normal to quick setting cement on account of heat was studied in various places. Cement specifications were subject of many changes in several countries and new standards were suggested. Relations between fineness and strength, type of storage and strength and between cement strength and concrete strength were investigated. Number of publications deal with investigations of influence of aggressive waters, salt solutions, acids, bases and organic substances on cement, mortar and concrete made of different kinds of cement and cement mixtures. Reactions, which take place when specimens are stored in above solutions, were carefully studied and conclusions on hardening process and durability were drawn. Improvements and new installations in modern cement plants are briefly discussed and several other miscellaneous research subjects mentioned. A total of 129 references are given.—A. E. BEITLICH

Closed circuit dry grinding improves quality of cement. *Rock Products*, April 26, 1930, V. 33, No. 9, p. 56-58.—Superior Portland Cement Co., Superior, Ohio, improved quality of cement and increased plant output 40 per cent by grinding in closed circuit with air separators. Plant is dry process using Bradley mills for preliminary grinding. Material is slag and limestone. After removal of iron by magnetic separator and grinding in Bradley mill material goes to four tube mills in closed circuit with air separators. Five were required before air separation was introduced and a 6-day week is enough to keep kilns supplied for 7-days. Air separators are 14 ft. diameter at top. The feed (tube mill discharge) contains 70 per cent of 200 mesh but much of it is sent back to the tubes. A heavy return has been found to increase grinding capacity and improve quality. Final product of air separators has 87 per cent through 200 mesh which is as good for burning as finer product. Burning is in five kilns. Clinker is coarsely ground in Bradley mill then in four tubes in closed circuit with two 16 ft. air separators. About 35 per cent of what is fed to separators is returned and 70 per cent of this passes 200 mesh. The final product is 95 per cent through 200 mesh. Power required is 35 h. p. for each 14 ft. and 40 h. p. for each 16 ft. air separator. Each tube is driven by a 175 h. p. motor. Output of plant is about 3250 bbl. per day.—EDMUND SHAW

Determination of heats of formation of silicates from their oxides. R. NACKEN. *Zement* (Germany), Aug. 28 and Sept. 4, 1930, V. 19, No. 35-36, p. 818-25, 847-9.—A method is described to determine heat of formation of meta- and ortho-silicates of alkali earth metals from oxides and silica. Heat of solution of silicates in mixtures of hydrochloric and hydrofluoric acid can be measured, and heat of formation can be calculated from obtained data. Reactions are carried out in calorimeter with water thermostat, which keeps outside temperature constant with an accuracy of 1:1000° C. Container is made of platinum and provided with shielded thermometer and stirrer. Specific heat of solvents was determined either by electrical method or by using heat of crystallization of an undercooled solution of salol. Heat of hydration of calcium hydroxide was determined very accurately. Silicates were prepared in electric furnace by heating exactly proportioned mixtures. Following substances were examined and their heat of formation determined: α-$CaSiO_3$ (pseudowollastonite), β-$CaSiO_3$ (wollastonite), δ-Ca_2SiO_4 (calcium ortho silicate), $SrSiO_3$, Sr_2SiO_4, $BaSiO_3$ and Ba_2SiO_4. Difficulties were encountered with barium compounds due to their insolubility, and it was necessary to use pure hydrofluoric

acid instead of a mixture of HCl and HF. Obtained results with barium were not very satisfactory. Attempts were made to study magnesium silicates but without success on account of very slow reaction. Lead silicates were dissolved in mixtures of nitric and hydrofluoric acid and their heat of formation was determined. Formulas for necessary calculations are given, and application of graphic method is described. Reactions which take place in clinker during its formation can be studied under use of above method. (cf. W. A. Roth, *Zement* (Germany), July, 1930, V. 19, No. 27, p. 628-30.)—A. E. BEITLICH

Modern tendencies in the manufacture of cement and inherent dangers. ANGELO LILI. Nomograph, Bologna (Italy), March, 1930, 5 p.—Some of attempts made since 1927 to change requirements of national law in Italy regarding cement strength and density have been toward increase in 28-day compressive strength requirement for "normal" mortar from 3980 lb. per sq. in. to 4960 lb. per sq. in. and a reduction in density limit from 3.00 to 2.90. It is cited that the better cements usually have density of about 3.18 and that only reason for going below 3.00 would be to permit dilution of cement by inert materials. Demand for high strengths at 28 days is difficult to meet by all manufacturers without the use of methods or materials that may be injurious to durability of cement. Best composition of cement with given raw materials is that which has just sufficient lime to combine with the silica and the iron and aluminum oxides. Any other composition will be likely in time for form unstable compounds. The cements of proper composition (hydraulic modulus not exceeding 2) show large increases from 7 to 365 days, and even with the tensile strengths, about 1/10 of the compressive strengths. High hydraulic modulus gives high early strengths, but relatively small increases between 7 and 28 days, and sometimes retrogression thereafter. Instability of volume and low tensile strength also occur with a high modulus. Author urges the State to make investigation covering all types of cements produced, and from that data to set up only important requirements. He states trend toward high strengths is absurd because the result is use of more steel, formation of more cracks, more chance of corrosion of steel, and much question as to durability of concrete. It is suggested that cements meet a requirement for "normal" mortar of 256 lb. per sq. in. in tension, and 2560 lb. per sq. in. in compression at 7 days, with appreciable increase at 28 days and beyond. Also that every bag of cement sold bear a diagram showing the normal strength—age curve for the cement, and name of manufacturer, addition of inert material to be prohibited. Specification based on the 7 day test is also proposed as follows:

$$N = \frac{f_7 \text{ (compressive strength of normal mortar at 7 days)}}{7},$$

28 day strength $= \frac{N}{a} \times 28$, 365 day strength $= \frac{N}{b} \times 365$, with suggestion that $a = 3$ and $b = 20$. Strength in tension at 7 and 28 days should be about 1/10 of the compressive strength. The advantages claimed for this specification are that this allows manufacturers to obtain best combination of raw materials and cements of highest value, for their conditions and provides a minimum strength requirement that will meet average demand, but which does not restrict production of higher strength cement.—C. G. CLAIR AND M. N. CLAIR

Blast furnace slag cements. A. O. PURDON. *Rev. materiaux construction trav. publics* (France), June, 1930, No. 249, p. 205.—American readers will find

definitions of: (a) "iron portland cement" (ciment portland de fer) as a mixture of a minimum of 70 per cent portland cement clinker with granulated blast furnace slag; (b) of "blast furnace slag cement" (ciment de haut fourneau) or "slag portland cement" (ciment portland de laitier) as mixtures as above containing lesser percentages of portland cement clinker. These are not to be confused with "slag cement" proper (ciment de laitier), a mixture of slaked lime and slag without addition of clinker. The article is a summary of test evidence showing that strength of blast furnace slag cements is equivalent to that of portland cement after a sufficiently long time interval. Clinkers of different origin were mixed in laboratory with various slags. The "ideal" proportions were computed making all of free lime in clinker become saturated by acid components (silica and alumina) of the slag. Uniform fineness was insured by grinding the two constituents separately before mixing. Tests show that "ideal" proportions are actually most advantageous. Admixtures of clinker exceeding this optimum (34.7 per cent) result in no further increase in strength, but act to reduce it. Commercial blast furnace slag cement was compared with standard portland and rapid-hardening portland cements, mostly of Belgian origin. The tests showed that, while during the first days slag cement lags considerably behind portland, at 3 mo. their strength values are very close and at 1 year almost identical. Tests at ages of 5 and 10 years are scheduled. Comparison is made with strength tests at 2 years of iron portland cement made in Germany by Tetmajer and Michaelis. The "iron portland" cements were recognized as equivalent in strength to standard portland in 1909 after a bitter struggle of 8 years. Objectors to "blast furnace slag" cements have emphasized the presence of calcium sulfate in slag with consequent danger of liberation of sulfuric acid. This acid, however, becomes immediately neutralized by lime continually formed during the setting and hardening of cement. Another argument relative to expansion of blast furnace slag cements is not confirmed by actual experience, these cements showing in a high degree the desired constancy of volume. According to Fritsch, the blast furnace slag cement shows remarkable resistance to saline waters, in which property it excels the standard portland cement. Finally, its low cost, due to the almost negligible cost of slag entering into mixture to extent of 70 per cent, makes it a most desirable product.—M. A. CORBIN

MISCELLANEOUS

A high-sensitivity absolute-humidity recorder. CRANDALL J. ROSECRANS. *Ind. Eng. Chem.*, Analytical Edition, V. 2, No. 2, p. 129-134.—An apparatus has been developed for measuring and recording the amount of water vapor in the air, which is suitable for recording humidities from zero up to saturation within a temperature range from 100° to about 200° C. This apparatus is based on measurement of the thermal conductivity of water vapor and air mixtures and is continuous reading.—ROY N. YOUNG

Tricalciumsilicate. RUDOLF BRILL. *Zement* (Germany), Aug., 1930, V. 19, No. 34, p. 796.—Tricalciumsilicate was prepared by fusion of mixture of CaO and SiO_2 in right proportions and keeping product at temperature of 1500° C. for 3¼ hours and following quenching. Comparison of its X-ray diagram was made with X-ray diagram of dicalciumsilicate and complete similarity was found. Author claims that tricalciumsilicate does not exist, and obtained product is only mixture of dicalciumsilicate and free lime.—A. E. BEITLICH

A stucco that did not harden. OTTO FRIZ. *Tonind Ztg.* (Germany), 1930, V. 54, No. 65, p. 1070-1.—A row of houses built of lightweight concrete block of pumice stone (Schwemmsteine) was covered with stucco, the first coat being a white lime mortar containing a little portland cement and applied to the well wetted walls. The second coat was a gray or dolomitic lime mortar.

Stuccoing was done in the fall and damp weather of late fall and early winter prevented the well soaked walls from drying so that the lime did not harden until a warm spell of weather in February permitted moisture to evaporate.—F. O. ANDEREGG

Contribution to the knowledge of ceramic properties of lime—alumina—silicates and refractory and high refractory materials. E. BERL AND FRITZ LOEBLEIN. *Forschungsarbeiten auf dem Gebiete des Ingenieurwesens* (Germany), 1930, No. 325, 28 pp.—Investigations were carried out to study softening and melting points of pure chemical melts and refractory bricks, composition of which falls in the system CaO—SiO_2—Al_2O_3. Special consideration was given to heat conductivity. Diagram is given showing relation between softening point and melting point and chemical composition. Graphs show compounds with like properties in three-component system. Mechanical strength depends very much on silica contents. Method is derived which allows determination of true heat number, which factor expresses heat conductivity.—A. E. BEITLICH

Types of minerals in artificial melts of $2CaO.SiO_2$, $3CaO.SiO_2$ and $8CaO.2SiO_2.Al_2O_3$. KATSUZO KOYANAGI. *Kogyo Kwagaku Zasshi* (*J. Soc. of Chem. Ind.* (Japan), May, 1930, V. 33, Supplemental binding, No. 5, p. 161.—Author prepared following melts: $2CaO.SiO_2$, $3CaO.SiO_2$ and $8CaO.2SiO_2.Al_2O_3$ from purest raw materials and studied their thin sections under microscope with polarized light. In all specimens an abundant amount of crystal needles with strong double refraction was found, especially in the $2CaO.SiO_2$ compound. Needles are believed to be dicalcium silicate. In $3CaO.SiO_2$ crystals were found with weak double refraction, corresponding with "alit." Free lime crystals were found under microscope in melts which showed great amount of free lime by glycerol method.—A. E. BEITLICH

Corrosion of Lead in Buildings. *Engineering* (England), April 11, 1930, p. 494.—Lead is usually resistant to corrosion since the surface exposed to moisture, oxygen and carbon dioxide is converted into a protective film of basic lead carbonate (mixture of lead carbonate and hydrate) but if CO_2 is lacking lead oxide is dissolved or if CO_2 is present in excess the oxide film is changed to soluble bicarbonate, and in either case corrosion results. Instances of failure have occurred caused by contact of lead with cement. Corrosive action of cements and mortars in presence of water, oxygen and carbon dioxide appears to depend upon their content of free lime, which absorbs carbon dioxide preventing formation of protective film of basic carbonate, and partly as a solvent exposing fresh surfaces for attack. Aluminous cements, containing no free lime show no progressive attack on lead. Old cement and mortar, the lime of which has been completely carbonated is also inert. Tin coatings offer greater resistance to attack than leads. To protect lead from corrosive action of cement of mortar, good practice is to provide a protective coating of bitumen.—G. M. WILLIAMS

Studies on fundamental synthesis of calcium aluminates and their hydration VI. SHOICHIRO NAGI AND RYUICHI NAITO. *Kogyo Kwagaku Zasshi* (J. Soc. *Chem. Ind.* (Japan), Jan., 1930, V. 33, Supplemental binding No. 1, p. 37-43.—Test data are given, which show rate of combination when mixture of $CaCO_3$ and Al_2O_3 in ratio of 3 CaO to 1 Al_2O_3 is heated. Comparisons are made with data obtained by preparation of $5CaO.3Al_2O_3$, $CaO.Al_2O_3$ and $3CaO.5Al_2O_3$, which were reported in previous article (*Kogyo Kwagaku Zasshi* (Japan), 1929, V. 32, p. 965). When a mixture with ratio 5CaO to $3Al_2O_3$ is heated, it tends to form $3CaO.Al_2O_3$ with increasing temperatures.—A. E. BEITLICH

The bauxite deposits of the Ariege. V. CHARRIN. *Genie civil* (France), Sept. 20, 1930, No. 12, p. 281.—Ariège bauxites are least accessible deposits,

being located in foothills of the Pyrenees. The stratum runs east and west and seems to be result of tremendous upheaval. The average width of workable stratum is about 3.4 miles. Its thickness varies from a few inches to from 20 to 25 ft. White, red and all intermediate shades are represented in this deposit. The analysis of red low-silica bauxite is: alumina, 62 to 65 per cent; silica, 6 to 7 per cent; iron oxide, 12 to 15 per cent. Low-alumina refractory bauxite has following analysis: alumina, 40 to 44 per cent; silica, 36 to 40 per cent; iron oxide, 1 to 3 per cent. The average run of bauxite has following quality: alumina, 50 to 55 per cent; silica, 15 to 18 per cent; iron oxide, 10 to 15 per cent.—M. A. CORBIN

Studies on fundamental synthesis of calcium aluminates and their hydration IV. SHOICHIRO NAGAI AND RYUICHI NAITO. *Kogyo Kwagaku Zasshi* (*J. Soc. Chem. Ind.* (Japan), May, 1930, V. 33, Supplemental binding, No. 5, p. 164-6.—Authors report further studies on three eutectic mixtures: (1) 47 per cent CaO:53 per cent Al_2O_3 which is eutectic between $5CaO.3Al_2O_3$ and $CaO.Al_2O_3$; (2) 2CaO:1 Al_2O_3 which is nearly equal to eutectic mixture of 50 per cent CaO:50 per cent Al_2O_3 and (3) 33.5 per cent CaO:66.5 per cent Al_2O_3 which is eutectic between $CaO.Al_2O_3$ and $3CaO.5Al_2O_3$. Temperatures are given at which combination of these compounds is completed. Compound $2CaO.Al_2O_3$, which was reported as pure aluminate is now believed to be mixture of $3CaO.Al_2O_3$ and $5CaO.3Al_2O_3$. Compressive strength was studied of obtained products by small-piece testing method proposed by authors. Further investigations of compounds: "alit," "celit" and "gehlenit" are under way.—A. E. BEITLICH

Action of salts, acids and organic substances on cement and concrete. RICHARD GRUN. *Zeitschr. angew. Chemie* (Germany), June 14, 1930, V. 43, No. 24, p. 496-500.—Investigations were carried out to study influence of solutions of different concentrations of acid salts, free acids and organic substances on setting time, compressive and tensile strengths of portland cement and blast furnace slag cement. Concentrated solutions of calcium chloride accelerate setting process, but with increasing dilution, setting time becomes longer for both cements. Ferric chloride, aluminum chloride and mixture of calcium chloride and aluminum chloride have similar effects. Barium chloride has little influence. Initial tensile strength is usually lowered, but will be normal at 28 days. Compressive strength is only lowered by concentrated solutions, while diluted solutions improve strength, especially after 28 days. Hydrochloric acid leads to shortest setting times with great evolution of heat. With concentrations of 1.1 and 1.8 per cent, tensile strength is lowered in most cases but compressive strength shows great increase. Organic substances slow up setting process and destroy nearly all strength properties. Hardened concrete test pieces of 1:3 cement—sand mixtures were stored in phosphoric acid and oxalic acid up to one year. Concentrations of phosphoric acids were 5, 10 and 15 per cent, oxalic acids 1 and 5 per cent. Both cements passed tests fairly well. No visible effect could be observed after six months storage in 5 per cent phosphoric acid. Strength decreases after this period. Stronger acids have destroying influence after short time. Oxalic acid shows in general no influence, 5 per cent concentration causing small decreases in strength. Normal strength was noted after one year. Several references are given.—A. E. BEITLICH

The Italian cement, lime and gypsum industries. (See MATERIALS—Cement.)

New dry paint for the concrete brick industry. (See SHOP MANUFACTURE.)

PROPERTIES OF CONCRETE

Remixing cement mortar—building research station. *Builder*

(England), Sept. 12, 1930, V. 139, No. 4571, p. 440.—Results of tests on retempering are given, and it is stated that although with portland cement it may be possible to regauge the mortar after a period up to four hours, with rapid hardening cement a definite loss of strength is indicated. Further if a wet mix were allowed to stand and evaporation of excess moisture resulted, a slight increase in strength for this reason alone, might be registered. The practice in general is not recommended.—JOHN E. ADAMS

New fundamental questions concerning concrete composition. Abram's aggregate modulus and Spindel's graphic concrete synthesis. OTTOKAR STERN. *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereins* (Austria), Aug., 1930, V. 82, No. 31-32 and 33-34, p. 255-7, 269-72.—Discussion of theory of four-component system and its application for calculation of concrete mixtures is given. Relation is shown between w/c-ratio and amount of dry solid substances. Formulas are given for calculation of voids and solids. In order to obtain best concrete consistency certain mixing rules are suggested which connect Abram's modulus, w/c-ratio and compressive strength. Limits for possible cement mixtures are calculated and directions are given for making of charts for above graphic calculations.—A. E. BEITLICH

The failures of mortars and concrete. "Les defauts des mortiers et des betons." J. MALETTE (France), Francs 63.50. Reviewed in *Der Bauingenieur* (Germany), May 9, 1930, V. 11, No. 19, p. 342.—First part discusses limes and cements, their constitution, chemical composition, behavior in sea water and specifications. Second part describes aggregates, mortar and concrete and illustrates all facts, which influence their normal behavior. Effect of aggressive waters and chemical substances on concrete is characterized. Numerous tests and experiments made by author conclude book.—A. E. BEITLICH

Chemical composition of outer layers of cement mortars and concrete II. SHOICHIRO NAGAI AND K. JOSHIWAZA. *Kogyo Kwagaku Zasshi* (J. *Soc. Chem. Ind.* (Japan), Jan., 1930, V. 33, Supplemental binding, No. 1, p. 43-8.—Experiments were made with mixed 1:2 mortars and w/c ratios ranging from 50 to 70 per cent. It was found, that loss on ignition and amount of calcium carbonate in outer layer increase with increasing w/c ratio. Sulphuric acid content increases simultaneously. Similar results were obtained with neat cements.—A. E. BEITLICH

Bond strength and resistance of concrete to impact and wear compared to its strength in compression, flexure and tension. R. FERET. 1930, Published by *Rev. materiaux construction trav. publics*, Paris (France), 1 V., 72 p.—The author distinguishes two fundamental classes of strengths: compression and shear, proportional one to the other, and tension and flexure, equally proportional one to the other, but showing no fixed relation with respect to the former. Personal experiments and detailed study of data from other sources show that in a series of mortars or concretes made with the same cement and differing by one variable factor only, tensile (or flexural) strength increases less rapidly than compressive strength and varies approximately as the 2/3 power of the latter. In mixes of varying composition, the relation of the former to the latter strength is approximately proportional to that of quantities of water and cement. This discrepancy between the two classes of strengths depends to a large extent on the fact that composition near the surface of test specimen is rarely identical with that of the core, due partly to unequal distribution of materials against the walls of the mold, and partly to unequal hardening of the mass, the curing medium having a more direct effect on some parts of the specimen. Aside from this, strengths are variably in-

fluenced by latent stresses, varying according to the distance from the exposed surface, and produced by changes in volume as a result of chemical, thermic and hygroscopic action. It thus appears that tests of concrete in tension and flexure do not show adequately the stresses of similar nature which concrete will withstand on the job, while compression strength values are proportional to actual strengths. Resistance to shock and wear, as well as bond strength differ from the fundamental classes of strengths, but appear in certain series of concrete tests to be proportional to one or the other group. Resistance to wear by abrasion depends almost entirely on the hardness of the rock materials used in the concrete.—M. A. CORBIN

Lactic acid, mortars and brick floors. E. SUENSON. *Der Bautenschutz* (Germany), 1930, V. 1, No. 1, 2, 4 and 6, p. 3-6, 30-3, 59-63, 80-5.—Comprehensive investigation was undertaken to study influence of lactic acid on cement mortars. Acid was used in three different concentrations: 1/10, 2/10 and 3/10 normal with pH:2.4, 2.3 and 2.2. Following kinds of cements were tested: (1) portland cement, (2) portland cement with admixture of Ceresit, (3) portland cement with Sika 1, (4) portland cement with Sika 4a, (5) Danish Velo cement, (6) French Alzement (high alumina cement) and (7) American Medusa White cement. Test pieces made for study of volume changes in mixtures 1:2 and 1:4 were immersed in pure water, three lactic acid solutions of above concentrations, milk and butter milk. No considerable changes in volume were observed. Solutions showed different grades of aggressive action on cements. Effect of hot and cold water on volume of test pieces was investigated. Strength tests were carried out with 1:4 mortars with small slabs and cubes which were stored in lactic acid solutions. Compressive and flexural strength properties were studied. Lactic acid lowers compressive strength much more than flexural strength. None of the special cements or mixtures showed much better behavior than ordinary portland cement. Resistance to acid and relation between compressive and flexural strength increases very much with cement contents and coarseness of sand.—A. E. BEITLICH

Action of salts, acids and organic substances on cement concrete. (See MATERIALS—Miscellaneous.)

Corrosion of lead in buildings. (See MATERIALS—Miscellaneous.)

Concrete of great soundness made up of portland cement and arsenious oxide. (See MATERIALS—Cement.)

Effect of addition of finer sizes of gravel on strength of concrete. See MATERIALS—Aggregates.)

Light weight concretes. (See MATERIALS—Aggregates.)

Use of under-water concrete in Sweden. (See FIELD CONSTRUCTION—Miscellaneous.)

ENGINEERING DESIGN

BRIDGES

Handbook for reinforced concrete. III. "Handbuch fur Eisenbeton." W. GEHLER. 1930, Wilhelm Ernst und Sohn, Berlin (Germany), V. 4, new edition, R. M. 6.80. Reviewed in *Der Bauingenieur* (Germany), May 23, 1930, V. 11, No. 21, p. 373.—Subjects of book are calculations and construction methods of reinforced concrete bridges, construction of roadways, joints, abutments, piers and hinges. Several outstanding bridge structures are discussed.—A. E. BEITLICH

Queen Margaret bridge, Glasgow. *Concrete Constr. Eng.* (England),

Sept., 1930, V. 25, No. 9, p. 510.—The bridge is a simple parabolic arch, having a skew span of 137 ft. between abutments. Angle of skew is 26 degrees. A subway is arranged over each abutment for traffic along the river. Some difficulty was experienced in the design of one of the abutments, owing to the presence of a 5 ft. brick sewer, which followed the line of the abutment beam. Difficulty was overcome by providing reinforced concrete beams over the sewer so horizontal component of thrust was taken over top of sewer. Reinforced concrete was used exclusively for the structural work and parapets of bridge were of red granite.—J. MARIN

Stresses under the Freyssinet method of concrete-arch construction. J. T. THOMPSON. *Eng. News Record*, Aug. 21, 1930, V. 105, No. 8, p. 291.—The principal advantages and economics claimed for Freyssinet method of concrete-arch construction may be briefly summarized as follows: (1) Elastic rib shortening and shrinkage stresses may be practically eliminated, with reductions in rib sections. (2) By postponing the keying until temperature of air has reached a desirable mean value, temperature stresses may be considerably reduced. (3) Secondary stresses due to lateral deflection of abutments may be practically eliminated. (4) Continuity may be developed at ends of centers in long spans, thus reducing deflections. Centers are struck by exerting jack pressures that widen the opening at the crown a sufficient amount to lift the rib clear of the centering. Author presents data relative to stresses in typical Freyssinet concrete-arch bridges in France where method has been used extensively and points out that highest stresses may be reduced as much as 35 per cent.—D. E. LARSON

BUILDINGS

A large reinforced concrete aeroplane hangar. C. E. HOLLOWAY. *Struct. Eng.* (England), Sept., 1930, V. 8., No. 9, p. 317-328.—A design in reinforced concrete, submitted in open competition with steel, was chosen for an aeroplane hangar having a clear floor area of 80 by 100 ft. and built at Heston Air Park, Middlesex, England. The design is featured by r. c. tied arches of 100 ft. clear span and 20 ft. rise, spaced 16 ft. on centers, and mounted on lead sheets. Main columns are provided with side wind struts which slope away from the tops of columns at 45 degrees. Wall panels were cast into preformed grooves in sides of wall members, thereby confining cracks to the grooves. Office and shop space is provided on the sides of the structure beneath wind struts and additional office space was made overhead between walls and adjacent arches, floors for the suspended offices framing into the arch ties. Analysis and details are given for arch ribs, hangers, ties, columns and struts.—V. P. JENSEN

Concrete airship hangar of bold design at Sevilla, Spain. A. A. PETERS. *Eng. News Record*, Aug. 28, 1930, V. 105, No. 9, p. 324.—Huge arched-roof hangar of reinforced concrete in unusual and bold design is being built for airships of Zeppelin type at Sevilla, Spain. Designed to hold two large dirigibles, it is 950 ft. long, 400 ft. wide and 200 ft. high. Plans were based on those developed for small war-time steel structure planned for Italy. It was proposed to utilize a portion of old steel shipped to that country prior to signing of armistice. This plan proved to be futile and cost study indicated that arch design in reinforced concrete would be cheaper than modern structural steel building. Structural steel centering is being used for arches and quick-setting cement is being imported from England to cut down time limit on removal of supporting forms and speed up cycle of operations for each setup. Cement gun is being used to place concrete in wall of bays between arch ribs. These walls are only 4 in. thick and contain double lines of reinforcing bars.—D. E. LARSON

DAMS

Circular concrete arch dam designed by simple method. R. P. V. MARQUARDSEN. *Concrete*, Oct., 1930, V. 37, No. 4, p. 27-29. Author outlines a method of calculating fibre stress in concrete of a fixed-end circular dam with a constant cross section in any horizontal plane. An elevation of the arch is drawn, a base line is established to permit proper application of the stress formula to the extrados and intrados sides of the various sections of the dam. The formula for fibre stress at any section involves water pressure, radius of curvature to both the center of the dam and the upstream face, thickness of dam or arch, coefficient of expansion or contraction, temperature changes, amount of settlement, modulus of elasticity of concrete, various angles including the central angle of the arch, and three sets of quantities which are functions of the central angle and which are given in three charts. Calculation of fibre stress for crown and end sections are very simple. Shearing stresses may be found by a simple formula submitted. Derivations of formulas given are to be published in November issue of Concrete.—N. H. ROY

MISCELLANEOUS

Nomogram for calculating slabs and beams, for calculating moments for different conditions of the supports. JORGE QUIJANO. Ingenieria (Mexico), March, 1930, p. 123 and 133.—Simple alignment or nomographic charts, of a series of such charts for use in reinforced concrete design, one solving the relation between M and A_s and d for given values of f_s, f_c and N. Left hand scale or line l middle scales A_s and d, and right scale W. All in metric units. The other chart solves the relation $M = CWl^2$. Left hand scale or line l, middle scale M, and right scale W.—C. G. CLAIR AND M. N. CLAIR

Slurry mixing tanks. *Concrete Constr. Eng.* (England), Sept., 1930, V. 25, No. 9, p. 513.—Reinforced concrete has been used exclusively at the Johnson's Works of the British Portland Cement Manufacturers, Ltd., for the construction of slurry mixing tanks. Details of tanks are given. Range in size is from 48 to 66 ft. internal diameter. They are 11 ft. 6 in. deep and the walls are 6 in. from top to bottom. Details of the design and of construction are given.—J. MARIN

Construction of reinforced concrete superstructures. "Practisches Konstruieren von Eisenbetonhochbauten." RUDOLF BAYERL AND ADOLF BRZESKY. 1930, Julius Springer, Vienna, 144 p., 67 illustrations, R. M. 7.00. Reviewed in *Die Bautechnik* (Germany), June 13, 1930, V. 8, No. 26, p. 418.—Design and calculation of superstructures are illustrated. Examples are included for different types of ceilings, columns, slabs, girders and frames. Influence of heat and shrinkage on structure is shown.—A. E. BEITLICH

Reinforced concrete coal bin. *Zement* (Germany), April 17, 1930, V. 19, No. 16, p. 386.—Structure has cross section of 46½ ft. square and capacity of 1200 tons. Total height is 120 ft. Bin rests on four closed frames, which are supported by eight vertical reinforced concrete columns. Foundation consists of continuous slab. Pumping station is located in basement of structure and pumps water to three water tanks on top of bin.—A. E. BEITLICH

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete* Sept., 1930, V. 37, No. 3, p. 35-37.—Chart A, given in Concrete, March, 1928, has been modified to allow higher strength concrete and higher working stresses allowed by the A. C. I. Code. Balanced tensile reinforcing for concrete beams may be had directly from the chart. The factors involved are breadth and

depth of beam, bending moment, steel area, and allowable stress in the concrete.—N. H. ROY

Statics of structures (in three volumes). "Die Statik der Bauwerke (in drei Baenden)." RUDOLF KIRCHHOFF. 1930, Wilhelm Ernst und Sohn, Berlin (Germany), 2nd V., 2nd edition, 368 p., 261 illustrations, R. M. 27.00. Reviewed in *Die Bautechnik* (Germany), June 13, 1930, V. 8, No. 26, p. 415.—Book deals with static calculations of form variations of structures. Construction of bridges (except suspension bridges) and of other kinds of superstructures is discussed. Beams, girders, columns and slabs are analyzed and their calculation illustrated. Ceiling and wall structures conclude book and third and last volume will appear very soon.—A. E. BEITLICH

Reinforced concrete shell roofs. H. SAVAGE. *Concrete Constr. Eng.* (England), Sept., 1930, V. 25, No. 9, p. 490-1.—Theory developed for the design of two kinds of shell roofs: shells curved in one direction and in two directions. Theory covers cases of unsymmetrical loading, any degree of fixation at the boundaries of the shell and any form of the shell itself. Approximate solutions are presented to simplify design. Experiments were made on large scale models to verify degree of accuracy attained in calculations by approximate methods. Main feature in design is reduction of bending moments in the shell. A graphical method of analysis is given for cases where the curve of the shell cannot be expressed mathematically, or shell varies in thickness, or load is complicated. Examples of various types of shell roofs are given. References for further information on shell roofs are cited.—J. MARIN

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete*, Oct., 1930, p. 39-40.—Professor Griffith presents Charts F_1 and F_2 for the determination of stirrup spacing when allowable stirrup stresses of 14,000 and 12,000 lbs. per sq. in. are desired. The formula for spacing conforms to the Joint Committee and A. C. I. Code. Only the excess of shear above 40 lb. per sq. in. is used in the charts. The charts involve breadth of beam, excess unit shear, stirrup spacing, and size of bars.—N. H. ROY

Economic designs of reinforced concrete walls and ceilings using "disk" effect. H. CRAEMER. *Zement* (Germany), July 24, 1930, V. 19, No. 30, p. 703-8.—A theory is derived which explains "disk" effect or mutual influence of neighboring structural parts on account of monolithic structure of concrete. Neutralization of certain stresses takes place. Economic importance is very evident in structures like silos and storage bins. Walls take up part of stresses and eliminate necessity of supporting beams. Diagrams show stresses occurring in such walls. A number of examples of practice indicate that more than 360 cu. yd. of concrete were wasted for construction of ten useless transverse beams in coal bin structure of average size. Other examples show "disk" effect in ceilings where horizontal stresses are taken up.—A. E. BEITLICH

New York subway construction. *Eng. News Record*, Aug. 7, 1930, V. 105, No. 6, p. 200-205.—New York City's third subway system, one of the great construction undertakings of the world is now more than 60 per cent completed. Construction and equipment cost of 55 miles of line will be $650,000,000. Virtually, design of each type of present subway structure is same as employed for corresponding type in older subways. The structure in open cut consists of transverse steel bents every 5 ft. with columns in wall lines and between tracks. Columns stand on concrete footings and concrete jack arches fill between wall columns and roof beams. Subway tunnel structure in rock consists of semicircular concrete arch over each track with row of columns between tracks carrying longitudinal girder against which opposed arches thrust. Soft ground and subaqueous tunnels are lined with cast iron segments with interlining of concrete to give smooth surface. Only exception to this

structure is ¼-mile shield tunnel lined with precast concrete blocks.—D. E. LARSON

Stress transmission in frictional-cohesive materials. R. W. H. HAWKEN. *J. Inst. Eng. Australia* (Australia), July, 1930, V. 2, No. 7, p. 247.—Author propounds and solves a general theorem of stress transmission in frictional—cohesive materials on the verge of movement, and correlates with it results of Rankine, Navier, Bell and Resal. In "general equation," giving the relation between principal stresses, cohesion and friction, it is usually assumed that one principal stress is represented in direction and amount by intensity of applied normal load, given by gravity in earthwork or by the testing machine in compression tests. For cohesive materials, author considers this untrue when movement is just about to take place. The potential tangential resistance to movement, due to cohesion is considered to be a supplementary shear stress introduced as a maximum on one plane in material with infinitesimal cohesion —that is to say non-cohesive material. This one plane will be critical plane in frictional cohesive material. Direction of principal stresses for assumed criteria of incipient movements is deduced by applying principle of component additions and in the ellipse deduced, the general equation is fulfilled. On critical plane, cohesion which is maximum potential resistance to shear, independent of normal pressures, and potential frictional resistance proportionate to normal stress, taken together, equilibrate the shear concomitant with the true principal stresses.—JOHN E. ADAMS

Statics of restrained and continuous beams. FRITZ EMPERGER. *Beton u. Eisen* (Germany), June 20, 1930, V. 29, No. 12, p. 216.—The moment curve can be exactly determined only for simply supported beams. Where restraint exists the moment variation may be studied by considering possible limiting conditions of restraint. Usual assumption of point supports, satisfactory in bridge work does not seem reliable for buildings where the columns act with the beams. Until recently all steel beams were designed for a moment of $\frac{wl^2}{8}$ throughout their entire length regardless of end restraint. The Austrian Reinf. Conc. Committee in 1908 (Heft 5, 1916) conducted some tests on restrained steel beams of 13 ft. spans. At that time restrained concrete beams were designed for a center moment of $\frac{wl^2}{10}$ while $\frac{wl^2}{8}$ was required for steel beams. The tests showed a completely fixed condition for steel beams with ends built in brickwork with portland cement mortar, and in concrete. Only partial fixity was found for brickwork with lime mortar. The Committee then recommended for steel beams so restrained, a resisting moment at center and at supports of $\frac{wl^2}{16}$. Later tests bear out 1908 investigation (c. f. Maier-Leibnitz, *Bautech.* 1929, Heft 20). Building departments of Leipzig and Hamburg have now approved this recommendation. Safety against failure should be governing factor in beam design. The sum of the center and support moments rather than moment at any one point is to be considered. Attainment of elastic limit at any point leads to a readjustment of moment curve, causing higher moments at other portions of beam. In concrete flexibility of design possible in reinforcement and haunching permits original moment distribution between center and supports to exist until failure. Tests deal with this problem. In tests by Austrian Reinf. Conc. Committee (Heft 4, 1913) a fully restrained concrete beam was reinforced for $\frac{wl^2}{24}$ at supports and $\frac{wl^2}{12}$ at center. At failure, the moments had adjusted themselves to fit this steel distribution. It is hoped to continue at some future date tests of duplicates of single span restrained beams already studied, when these beams are parts of rigid frames.—A. A. BRIELMAIER

A new construction principle. (See FIELD CONSTRUCTION—Miscellaneous.)

ROADS AND PAVEMENTS

Concrete roads—joints. *J. Inst. of Municipal County Engineers* (England), Sept. 2, 1930, V. 57, No. 5, p. 22.—The Ministry of Transport and the County Surveyor of Surrey are collaborating for the purpose of investigating the properties of different types of joints in concrete roads, and the most important relating to the experimental section of road are given.—JOHN E. ADAMS

Joints in concrete roads. *Concrete Constr. Eng.* (England), Sept., 1930, V. 25, No. 9, p. 523.—Study of failure of roads recently built in England shows the striking feature that in practically every case the defect is at a joint. A committee of the Ministry of Transport on Experimental Work is investigating the case and in conjunction with highway authorities concerned, intends to open up some of the joints and determine: (1) exact position of the concrete beams under expansion joints and how they behave; (2) what subsidence (if any) has taken place at joints; (3) characteristics of concrete where failures have occurred, and also to collect information as to any special features of drainage, height of embankments and flooding.—J. MARIN

WATER WORKS

Water tower in Stoppenberg (Germany) of 70,630 cu. ft. capacity. A. KONRAD. *Der Bauingenieur* (Germany), March 14, 1930, V. 11, No. 11/12, p. 199-202.—Entire structure is made of reinforced concrete. Tank has cylindrical form, 45.9 ft. inside diameter and 44.3 ft. high. Weight of tank (1430 tons) and live load (3300 tons) are supported by 9 inner columns, 2 ft. 5 in. square, and 12 outer columns, 2 ft. 2 in. by 2 ft. 4 in. All columns are connected by vertical stabilizing beams. Foundation consists of 2217 sq. ft. reinforced concrete slab, 3 ft. 3 in. thick. Underground is expected to sink slowly about 6 ft. on account of undermining by coal mine. Walls and bottom of tank are very heavy reinforced to stand high water pressures. Layers of insulating materials were placed inside of tank but were later removed and replaced by "Betonit" insulation, since small leakage of water occurred. Iron portland cement was used exclusively for entire structure. Its tensile strength was 498 lb. per sq. in.—A. E. BEITLICH

Water tower at Hertford. *Concrete Constr. Eng.* (England), Sept., 1930, V. 25, No. 9, p. 505-10.—The structure is situated in a growing residential district and, for this reason, attention was given to the question of appearance combined with economy. It was decided that appearance of a reinforced concrete tower would give concrete advantage over a steel structure. Details of design are given. Floor of tank is 75 ft. above ground level. Capacity is 200,000 gal. A feature of design was the dispensing of horizontal bracing for appearance by increasing size of supporting columns.—J. MARIN

North Bay's new water supply system. W. B. REDFERN. *Contract Record Eng. Rev.*, April 30, 1930, Vol. 44, No. 18, p. 495.—In connection with rebuilding of water supply system of North Bay, Ontario, a new concrete lined reservoir of 4,500,000 gal. capacity was constructed. Reservoir excavated in hardpan and rock is paved with 1:1½:3 mix reinforced concrete 8 in. thick. Lining of one side is supported on concrete buttresses due to insufficient depth of embankment. Bottom is underdrained, with tile lines and is divided into 45 ft. squares, expansion joints consisting of 12 in. copper strips and elastile.—G. M. WILLIAMS

Enlargement of a water tank in Schwaebisch Gmuend (Germany). *Zement* (Germany), April 17, 1930, V. 19, No. 16, p. 386.—Tank, which is 16 ft. high, 80 ft. long and 50 ft. wide has capacity of 1930 cu. yd. and is made of reinforced concrete. Top of tank is Pilz ceiling, and high early strength cement was used for its construction. Ceiling was calculated for load of 32 in. layer of ground and additional live load for tennis courts, which were placed on its top. Special consideration was given to perfect insulation of walls.—A. E. Beitlich

Architectural Design

Church of St. Konrad in Freiburg (Germany). C. A. Meckel. *Zement* (Germany), Aug. 28, 1930, V. 19, No. 35, p. 826-8.—Church is built of reinforced concrete in modern style. It is 65.6 ft. wide, 161.6 ft. long and provides room for 1900 persons. Total interior height is 47.6 ft., that of tower structure is 98.4 ft. Floors consist of concrete slabs, which are specially polished. Roof is vault construction resting on 7 cross beams, with no columns in interior. Use of reinforced concrete throughout makes building fireproof.—A. E. Beitlich

Modern questions of architectural design. "Die Zeitfragen der Architektur." Fritz Schumacher. 1929, Eugen Diederichs, Jena (Germany), R. M. 9.50. Reviewed in *Der Bauingenieur* (Germany), Jan. 3, 1930, V. 11, No. 1, p. 17.—Book belongs to most important publications in field of architectural design of modern structures. Different chapters deal with construction of buildings, bridges, towns and marine structures. Special consideration is given to application of colors in construction practice.—A. E. Beitlich

Field Construction

Bridges

Fourth Ave. viaduct, Moose Jaw, Sask. *Canadian Engineer*, June 24, 1930, V. 58, No. 25, p. 691-692.—To replace a timber trestle, City of Moose Jaw has built a mushroom flat slab viaduct consisting of 30 ft. roadway and two 5 ft. walks with spans of 30 ft. supported on concrete columns. Reinforcing of flat plate slab consists of ½ in. plain bars with a 6 in. spacing in two way mat and diagonal bands with a 3 in. spacing in bottom of plate. Concrete mixture was 1:2:4 by volume using a clean coarse sand and a crushed gravel. Consistency was such that mixture would flow slowly into place with little puddling or jarring. Tests at 34 days showed a strength of 4000 lb. per sq. in. Warm mixing water and ½ per cent of an admixture was used.—G. M. Williams

The Montreal South Shore bridge over the St. Lawrence River. *Eng. and Con.*, Sept., 1930, p. 330-336.—Substructure for South Shore bridge required 62 permanent piers with many types of foundations. A caisson 127 ft. 9 in. by 50 ft. 6 in. by 47 ft. was required for one main pier. Main pier on the city side was founded on two pneumatic caissons carried to rock. It is 47 by 96 by 150 and 200 ft. and contains 18,000 cu. yd. of concrete. River piers are of 2000 lb. concrete and are faced with limestone from a few feet below water level to a few feet above highest ice level. South approach required a number of piers. Ninety-two thousand cu. yds. of concrete were used in the substructure.—N. H. Roy

Concrete arch bridge carries pipe line. F. M. Preston. *Canadian Engineer*, May 27, 1930, V. 58, No. 21, p. 605-606.—Victoria, B. C., has re-

placed an old Howe Truss timber bridge with a reinforced concrete bridge designed to carry two 32 in. diameter steel water supply lines. Crossing 177 ft. long consists of five 16 ft. approach spans and an arch span of 66 ft. with ribs on 7 ft. centers. Arch is of the three hinged type, ribs being precast on their sides of a 1:1:2 concrete mixture and weighing 6½ tons each. Hinges in abutments and at crown consist of brass bushed steel pins turning in cast steel hinges. Approach trestles and abutments on rock were built first and the arch ribs erected by means of a floating derrick. One unit allowed to accidentally drop was recast with quick hardening cement and put in position four days later.—G. M. WILLIAMS

New construction of the Saxon bridge over river Elster in Leipzig (Germany). WESTPHAL. *Der Bauingenieur* (Germany), May 23, 1930, V. 11, No. 21, p. 359-61.—Bridge consists of 12 continuous reinforced concrete beams which rest on two shore abutments and two river piers. Opening between piers is 65.6 ft. and between piers and abutments 54.1 ft. Roadway is 39.4 ft. wide but can be widened to 52.5 ft. Side walks, 19.7 ft. wide are on each side. Larssen steel sheet piling was driven and excavations were made on each river side. One shore abutment and one pier were erected in one excavation. Pile driving was carried out under great difficulties, since underground was very soft. Two pumps were put into operation. Roadway consists of 8.7 in. thick concrete layer with asphalt top. About 24,700 cu. ft. concrete were used for its construction, and abutments and piers consumed 76,500 cu. ft.—A. E. BEITLICH

Quebec bridge roadway construction. JAQCUES JOBIN. *Canadian Engineer*, Aug. 26, 1930, V. 59, No. 9, p. 261-264.—The Quebec Bridge built by the St. Lawrence Bridge Co. and opened to railway traffic in 1917, carries two railway tracks 32 ft. 6 in. center to center, supported on four through plate girders, attached to girder floor beams. Space of 17 ft. 6 in. between interior plate girders was only portion available for roadway purposes. Steel floor beams and stringers attached to the plate girders carry reinforced concrete slab poured in place, which serves as wearing surface of roadway. To allow for expansion and contraction of supporting steel structure, expansion joints of overlapping plate type were provided to permit of a maximum movement as much as 3½ in.—G. M. WILLIAMS

The reinforced concrete arch bridge near Conflans-Fin-d'Oise. O. S. *Zement* (Germany), Aug. 1930, V. 19, No. 34, p. 804-7.—Bridge over river Oise near its junction with Seine river was constructed between March 1928 and August 1929, and was opened for traffic in December 1929. Arch has span of 414 ft. with a height of 55 ft. Cross-section of both arches is rectangular, 6 ft. 10 in. high in center and 4 ft. 2 in. high at abutments. Roadway is 16 ft. 6 in. wide with two 3 ft. 4 in. sidewalks. Vertical columns are spaced about 20 ft. on centers and have octagonal cross-section with 8 in. diameter. Columns are connected with cantilevered transverse slabs on which roadway is built. Latter consists of continuous concrete slab. Arches are hollow for 184 ft. from center on both sides. Wall thickness varies from 1 ft. 2 in. to 1 ft. 6 in. Description of reinforcements and their arrangement is given. Abutments were built upon concrete pile foundations.—A. E. BEITLICH

Handbook for reinforced concrete III. (See ENGINEERING DESIGN—Bridges.)

BUILDINGS

New buildings of the City of Karlsruhe (Germany). OTTO LINDE. *Zement* (Germany), Aug., 1930, V. 19, No. 33, p. 780-2.—Several new structures, including two school buildings, made of reinforced concrete are described. Special consideration is given to modern economic construction methods.—A. E. BEITLICH

Lightweight floors and concrete units in office building. *Concrete*, Sept., 1930, V. 37, No. 3 p. 21.—New type of floor panel—rivet grip steel deck construction—used in erection of 23-story building, has as supporting members steel bar joists, placed in the usual way on principal girders of structural frame and anchored at ends. At midspan a reinforced concrete bridging beam supplies a higher degree of lateral rigidity than otherwise possible. The ribbed steel pans are of 20-gauge metal and provide a solid, leak-proof decking or form for the 3-in. reinforced concrete slab placed over the deck. Exterior walls throughout the building are backed up with Haydite concrete units, standard, half and special size units, totaling 100,000 were used. Absorption test was made as follows on Haydite units and competing material: Four-inch slab of Indiana limestone, representing exterior facing material was placed flatwise in broad and shallow pan of water. Back-up units were then set in mortar, with the thickness of mortar joints between units. Water depth of 2 in. was maintained throughout. Four months later moisture had penetrated both slabs of limestone and the clay tile to practically full height, stopping at level about even with the uppermost shell. Penetration in Haydite units extended from 1 to 1½ in.—C. BACHMANN

Skeleton structures of reinforced concrete. G. RUETH. *Zement* (Germany), July 31 and August 7, 1930, V. 19, No. 31 and 32, p. 728-33, 755-8.—Use of high early strength cement for construction of lower parts of high skeleton structures is recommended in order to decrease dimensions of supporting columns as far as possible. Description of a number of such buildings illustrates application of this construction method for apartment and office buildings, water towers, industrial buildings, hospitals and warehouses.—A. E. BEITLICH

Rapid methods for reinforced concrete construction. WILHELM PETRY. *Der Bauingenieur* (Germany), Jan. 17, 1930, V. 11, No. 3, p. 35-9.—Modern methods for reinforced concrete construction enable erection of such structures with greater speed than was possible several years ago. Great 6-story warehouse in Kiel (Germany), which rests on deep pile foundation was finished in four months although underground work was done under great difficulties on account of high ground water. Addition to paper plant Scheuffelen in Oberlemmingen (Germany) (cf. *Zement* (Germany), June, 1930, V. 19, No. 24, p. 565-7) is 325 ft. long, 80 ft. wide and 68 ft. high. Its construction was delayed by bad weather conditions but was finished in 3 months. Several more examples are discussed and construction methods described.—A. E. BEITLICH

Modern methods and equipment used on American Can building. M. THOMPSON. *Concrete*, September, 1930, V. 37, No. 3, p. 13.—Mixing plant was designed to speed up the handling of both the raw materials and mixed concrete to place 700 cu. yds. daily on American Can Co. building. From bins sand and gravel were delivered to the three mixers by gravity, through weighing and measuring devices. Quantity of water, including any in aggregate, was about 7 gal. per sack of cement in column concrete and 7½ gal. for floor concrete. Proportions were about 1:2:4 for floor concrete and from 1:1½:3 to 1:1:2 for column concrete. Where 2,000 lb. concrete was required, strengths ranged from 4,284 to 4,810 lb. per sq. in. at 28 days. Batches were delivered to a skip at base of hoisting tower, elevated to hopper, then delivered to first of a series of portable belt conveyors, one being 52 ft. long, three each 40, two 37, and one 34 ft. long. Metal forms were used for columns, column heads, drop panels and floor panels. Drop panel forms for flat-slab construction were made in two parts, each containing half the semi-circular opening made by column head. Drop panel forms were supported from the story below by jack frames of 4-by-4 wooden posts and 4-by-6 jackheads, spaced 4 ft. apart. Floor panel forms were rectangular sheet metal panels.—C. BACHMANN

Foundations for library and convention hall of German Museum in Munich (Germany). E. STECHER. *Die Bautechnik* (Germany), March 18, 1930, V. 8, No. 12, p. 197-9.—Museum is located on island in river Isar. Foundations of structures consist of concrete piles. To reach firm stratum 520 vertical shafts were drilled to a depth from 33 to 66 ft. Special movable steel drills were put into operation. Shafts were conically enlarged at bottom. Concreting was carried out with plastic concrete without interuption until ground water level was reached, then upper part of foundations was finished. Concrete piles are connected with horizontal beams on which superstructures are erected. (cf. Hoffmann. *Zement* (Germany), May, 1930, V. 19, No. 20, p. 470-2.)—A. E. BEITLICH

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Sept., 1930, V. 37, No. 3, p. 27.—In assembling form work for a typical tower, forms are erected one story at a time and thoroughly braced with diagonal braces of 14 x 6 in. material and 3 x 6 in. bracing placed horizontally. Horizontal diagonal wires are twisted across the tower form work just above and below the horizontal 3 x 6 in. bracing. Form work for a story is supported at the corners by "Ell" blocks. Double waling strips serve at main holding members. Waling strips are extended along full length of tower on outer sides when there is only a small offset between face of column and face of wall.—C. BACHMANN

The future of reinforced concrete without forms. G. HOFFNER. *Le Constructeur Ciment Armé*, June, 1930, No. 129, p. 136.—Article urges use of steel and reinforced concrete type of construction conforming to American practice. Standardization of units will permit extensive use of precast units and will eliminate wasteful timber forms, replacing them by standard metal molds. A reinforced concrete building comprising about 12000 sq. yd. of floor area was erected in accordance with this system by Michelin plant at Clermont-Ferrand, the entire job taking only 6 months for its completion. Foundations were finished by end of June 1929, the skeleton and floors by October 1929. About 1200 sq. yd. of floor, including supports and beams, were placed every 10 days. The tests of the finished structure yielded satisfactory results. Other buildings of this type in France are: the auditorium of hotel at Hermitage, hotel Majestic at Clermont-Ferrand and the Renault garages at Saint-Etienne.—M. A. CORBIN

Reinforced concrete structure of new storage and warehouse of the Engelhardt brewery in Berlin (Germany). FR. HERBST. *Zement* (Germany), Aug. 14 and 21, 1930, V. 19, No. 33-34, p. 776-9, 800-4.—New building is located on Lake Rummelsburg near Stralau and was erected from fall 1929 to spring 1930. It is a reinforced concrete skeleton structure, with 96,880 sq. ft. floor area and is 192.3 ft. long and 126.3 ft. wide. Room of 192.3 by 45.9 ft. in basement is taken up by 8 decarbonation tanks with total capacity of 11,000 cu. yds. Total height of building is 98.4 ft. Of special interest is heavy insulation of walls and floors of basement and first floor to maintain constant low temperature. Difficulties were encountered in placing foundation on account of ground water. Foundation is one continuous reinforced concrete slab, is 3 ft. 3 in. thick under 6 and 4-story part of building and 2 ft. thick under 2-story part. Ground water was lowered by means of 20 wells. Basement floor is only a few inches above normal ground water level. Stairways were made of reinforced concrete. Pilz construction was used for ceiling slabs calculated for heavy live loads. Slabs from 8.66 in. to 18.90 in. were supported by square or octagonal reinforced concrete columns, diameters of which increased from 2 to 3 ft. Two concrete mixers delivered material to two distribution towers, 118 and 190 ft. high. Aggregates were transported with dump cars from boat to mixer and cement was added to aggregates in dump cars. Total construction consumed 814 tons high early strength cement, 1360 tons portland cement, 250,000 cu. ft. gravel and 745 tons reinforcing steel.—A. E. BEITLICH

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, August, 1930, V. 37, No. 2, p. 31-34.—Author describes a case where a comparatively thin panel wall of reinforced concrete occupies entire space between wall columns or pilasters, and between the floor slab and wall girder supporting floor above. A stub wall is concreted at the time wall girder and floor slabs are concreted. A key left in top of stub helps to unite it with fresh concrete placed upon it. At top the panel wall forms are held in alignment by U-shaped yokes. Wedges or blocks are inserted between yokes and face of column or pilaster to adjust form work horizontally. The inner and outer panel wall forms are held in proper relation to each other by spreaders and tie bolts and a pair of waling timbers at the top. At the bottom, just above the stub wall, there is another pair of waling timbers, or rangers, made of either single 3-by-4's or of two 1-by-6's or two 2-by-6's. Line drawings show details of form work for spandrel or upturned wall beam itself and how form work is connected with form work for wall column or pilaster. Details are application to a building of flat-slab type of reinforced concrete construction in which window area is to be as large as possible. Since the window extends up to underside of floor slab, the wall space immediately above, between floor slab and window sill, must serve both as a wall and as a girder. Since the spandrel or upturned beam is to be continuous with the floor slab, the inner panel of form work must be left open and unobstructed where slab occurs. Here the form work is supported on small precast blocks of concrete whose height is equal to the depth of slab, these blocks being left in place.—C. BACHMANN

Concrete airship hangar of bold design at Sevilla, Spain. (See ENGINEERING DESIGN—Buildings.)

DAMS

Aggregate for Boulder Dam. *Rock Products*, April 26, 1930, V. 33, No. 9, p. 54-55.—John Mocine, Gen. Sup. of United States Lime Products Corp., San Francisco, Calif., proposes novel method of quarrying large dolomitic limestone deposit for producing the 10,000 tons per day of aggregate that will be needed for Boulder Dam. Method consists in driving tunnel 280 ft. back of and parallel to quarry face. Twenty parallel tunnels are to be driven from the main tunnel to within 40 ft. of the face where each terminates in cross tunnels to be filled with powder. When this is shot a slab 40 ft. thick and height of bank will be brought down. Loading and stemming for next shot can go on without waiting for face to be cleared.—EDMUND SHAW

Transportation methods for raw materials for construction of Grimsel dam (Switzerland). W. FRANKE. *Der Bauingenieur* (Germany), March 14, 1930, V. 11, No. 11-12, p. 192-8.—Site of dam is located about 6000 ft. above sea level and transportation of structural materials was extremely difficult. Cement was delivered by rail to aerial tramway 10.5 mi. long which brought materials to construction site. Dam consists of two sections which are separated by natural rock of Mount Nollen. First section will be 377.3 ft. high and 590.5 ft. wide at its base; 445,000 cu. yd. concrete are necessary for its construction. Second part of dam will consume more than 74,000 cu. yd. concrete. Detailed description is given of aerial tramway and concrete distribution arrangements. Two cable cranes which handle 5600 to 6300 cu. ft. concrete per hr. are used and up to 4,000 cu. yd. concrete were placed in one day. Interesting details of construction of three power plants, connected with dam, are given.—A. E. BEITLICH

Power development on the Magog River at Sherebrooke, Que. VERNON R. DAVIES. *Canadian Engineer*, April 29, 1930, V. 59, No. 17.—Sherbrooke Land and Water Power Co. at Sherbrooke, Quebec, has completed a hydro-power development on the Magog River consisting of a concrete dam,

concrete lined head race canal and power house. Installation consists of 2-900 K. V. A., 2400 volt generators driven by 1100 h. p. water wheels with open flume and propeller type runners. Dam foundation consists of rock from which weathered material was removed and a cut-off trench excavated for the entire length of bulkhead and sluiceway sections. Bulkhead portion is a gravily section having maximum height of about 30 ft. To increase stability dowels, 1½ in. rods 10 ft. long, were grouted 5 ft. into the foundation rock on 2 ft. centers near the upstream face. Tests made to determine effectiveness of these dowels indicated they were of doubtful value. Head race canal 15 ft. wide and 15 ft. deep is lined with reinforced concrete walls 2 ft. thick above the sides of the rock excavation.—G. M. WILLIAMS

Power station Kriebstein near Waldheim (Germany) on river Zschopau. KIRSTEN. *Die Bautechnik* (Germany), March 14, April 4 and 11, 1930, V. 8, No. 11, 15 and 16, p. 149-53, 233-7, 247-50.—Big concrete dam was erected to produce power and to protect low lands against high water. Detailed descriptions of four different projects are given. Site of plant is very favorable in narrow valley of river Zschopau. Bedrock consists mostly of granite with a 9 to 21 ft. top layer of soft yellow loam and sand, which was partly removed. On left hand river bank cement was forced into rock voids to make ground impermeable to water. Quarry, located about half a mile upstream, furnished aggregates of high quality which were crushed in special plant. Dam is of overflow type, 787.4 ft. long and 13.1 ft. wide at its crest, radius of curvature is 73.8 ft. Middle part of dam is built as 85.3 ft. wide weir, top of which is 13 ft. below normal water level of reservoir. Overflow is divided into 6 sections, 22.3 ft. wide, which are separated by six 8.2 ft. wide pillars. Height of dam is 86.3 ft. and it is 66.9 ft. wide at base. Joints were made at distances from 65 to 82 ft., sealed by means of sheet copper embedded in bitumen and asphalt. Great number of thermocouples recorded heat developed in interior of dam. Overflowing water is collected in basin 9.8 ft. deep and 105 ft. long where its force is broken. Floor of basin consists of 6 ft. thick concrete slab with 1 ft. top layer of reinforced concrete. Reinforced concrete bridge designed for load of 40.8 lb. per sq. ft. is built over basin to gates at base of dam. Its span is 114.8 ft. Dam is provided with 3 openings at its base, each 8.5 ft. in diameter and being capable to take in 3355 cu. ft. of water per sec. Gates are operated from top of dam. Three 118 ft. long high pressure pipes lead water to 3 turbines. Diameter of two of those pipes is 7.9 ft., third pipe for small turbine is 5.9 ft. Power station is reinforced concrete skeleton structure with 3 turbines. Structural materials were stored on left hand side of river. Cement, trass and sand were transported by cableway to central mixing plant and silos. Four towers distributed concrete. 1020 cu. yd. was placed daily. Two concrete mixtures, consisting of 440 lb. portland cement and 165 lbs. trass and 396.8 lbs. portland cement and 143.3 lb. trass for 35 cu. ft. concrete were used. Water impermeability of concrete was very much improved by addition of trass. Aggregates consisted of 15 parts sand (0 to ⅛ in.), 15 parts sand (0 to ¼ in.), 41 parts gravel (¼ to 1¼ in.) and 29 parts gravel (1¼ to 2½ in.) Laboratory at site of construction tested strength and water permeability of all mixtures.—A. E. BEITLICH

MISCELLANEOUS

Repairs of Bildwasen tunnel (Germany). K. SCHAECHTERLE. *Die Bautechnik* (Germany), April 25, 1930, V. 8, No. 18, p. 271-3.—Railroad tunnel of 1880 ft. length was badly damaged by water and engine smoke. Rock layer on top of tunnel was partly removed and replaced with concrete filling. New drainage system was built.—A. E. BEITLICH

Guniting sea-water structures. *Concrete Constr. Eng.* (England), Sept., 1930, V. 25, No. 9, p. 521.—Article refers to various wharf constructions to

point out that experience shows importance of ensuring that top surface of the slab be made water tight. Special methods are given with preference to method of applying grout by cement-gun to super structure and under side of the deck. This largely reduces the rusting of the steel reinforcement. Examples given to illustrate the success of the method.—J. MARIN

Construction of coal bin in Duisburg Ruhrort (Germany) with 11,000 tons capacity. HEINRICH BUTZER. *Der Bauingenieur* (Germany), March 14, 1930, V. 11, No. 11-12, p. 190-2.—Coal silo structure consists of 32 cells; it is 367.5 ft. long, 45.9 ft. wide and 55.8 ft. high, with capacity approximately 11,000 tons. Cells are provided with conical bottom, 22 (for coarse coal) show an angle of 50 degrees and 10 (for fine coal) are built with angle of 65 degrees. Unfavorable underground conditions made foundation on reinforced concrete piles necessary. Entire structure is divided into 3 parts by 2 expansion joints.—A. E. BEITLICH

Successful transit mix concrete. R. M. BELL. *Rock Products*, June 7, 1930, V. 33, No. 12, p. 109.—Brief description of operation of Swigert, Hart and Yett, Portland, Ore., where portable charging plant was introduced to shorten the haul on a 60,000 yd. job. Not many details are given.—EDMUND SHAW

Winter construction. A. SUTTER. *Der Bauingenieur* (Germany), March 7, 1930, V. 11, No. 10, p. 160-3.—Best way to make construction work possible during severe cold weather periods is to protect whole structure against cold outside air. Then the temperature can be kept within normal range by heating with hot air. Raw materials must be preheated. Addition of anti-freezing agents to concrete is not advisable, since this mostly causes considerable loss in strength. From economic view point winter construction must be highly recommended. Costs for heating and protecting amount to only a small fraction of wages, and work is provided for laborers during cold weather seasons.—A. E. BEITLICH

Notes on concrete and reinforced concrete made on trip through United States of America to Eastern Asia. E. PROBST. *Der Bauingenieur* (Germany), March 14, 1930, V. 11, No. 11-12, p. 180-6.—Author gives his impressions gathered on trip through U. S. A., Japan and China. He discusses modern American methods for concrete road and dam construction, use of lightweight concrete and application of concrete for railroad construction jobs. Economic conditions for cement and concrete in Japan and China are characterized and their new standard specifications and methods are criticised.—A. E. BEITLICH

Construction of a waste water sewer pipe system under river Rhine near Koeln (Germany). OTTO WERKEN. *Die Bautechnik* (Germany), March 7, April 18 and May 2, 1930, V. 8, No. 10, 17 and 19, p. 137-9, 261-4, 285-8.—Increasing population of city of Koeln made construction of new sewage disposal plant necessary. New structure is located on right side of river Rhine. Sewage from old plant on left side had to be led underneath river to new plant. Two sewer pipes, 6.1 ft. and 3.1 ft. in diameter, capable of carrying 425 to 450 gal. of water per hr., were built in one single concrete body. Pipes having total length of 1119 ft. were laid in excavation in river. Connections were made under water by means of diving bells. Detailed description of interesting placing method and several illustrations are given.—A. E. BEITLICH

Tall concrete chimney construction. *Canadian Engineer*, June 17, 1930, V. 58, No. 24, p. 683-684.—A concrete chimney 422 ft. 6 in. high and 8 ft. in diameter has been erected by the Horne Copper Corporation, Noranda, Quebec. Base on solid rock consists of a concrete ring 7 ft. 6 in. deep, 35 ft.

outside diameter and 23 ft. inside diameter, reinforced top and bottom with rings of 1 in. bars and ¾ in. radial bars. Concrete shaft was designed to resist combined loading of its dead weight and a wind pressure of 25 lb. sq. ft. of projected area. Connection of shaft to base is made by means of 206 1 in. steel dowels and shell is reinforced by both vertical and horizontal steel. To withstand acid action of gases stack is lined with a 4 in. acid resistant, vitreous brick laid up in special acid proof mortar, built in 45 ft. sections supported on ledges which are protected by a lead flashing where exposed. In addition inside surface as well as the outer 100 ft. of the concrete surface of the shaft is covered with an acid resistant bitumastic paint. Concrete mix for base was 1:2:4 cement, sand and gravel and for shaft a 1:2:3, with average slump of 6 inches. Adjustable all steel forms supported by an interior scaffold were used.—G. M. WILLIAMS

American foundation methods. H. GRIESEL. *Der Bauingenieur* (Germany), Feb. 7, 1930, V. 11, No. 6, p. 91-6.—First part of article describes foundations with wooden piles, concrete piles, reinforced concrete piles, Raymond piles and combinations of wood and concrete piles. Advantages of Raymond piles are especially characterized. Pile driving and necessary equipment are illustrated, and examples of practice are shown. Second part deals with caisson methods. Caissons can be placed either by excavating or by means of compressed air. Foundation job during construction of Delaware bridge between Philadelphia and Camden is described with dimensions and detailed graphs. Construction of Holland tunnel under Hudson river is example of foundation with compressed air method. Several foundations of skyscraper buildings are illustrated.—A. E. BEITLICH

Winter concreting methods applied on contract of moderate size. ROBT. C. JOHNSON. *Concrete*, Oct., 1930, V. 37, No. 4, p. 13-16.—Concrete plant on 4,000 cu. yd. job of wall-bearing design was limited to concrete mixer. Responsibility for quality depended upon competent men and mechanical controls were eliminated. Aggregates were measured by wheelbarrow, and 8 to 10 test samples were taken daily. Proportions were 1:2:4. One man was detailed through winter months to take three test cylinders from each sample of concrete, to be broken at customary periods, and to keep temperature records. Specifications contained two clauses: temperature of 60 to 80° for concrete being poured, and that freshly placed concrete be kept at minimum of 70° for at least five days after concreting. Following equipment was used for protection: Vertical steam boiler with 100 lb. pressure capacity; 150 lin. ft. steam hose, about 100 salamanders, canvas, 10,000 sq. ft.; Sisalkraft, 20,000 sq. ft. Total cost of material used directly for winter work and winter work labor costs amounted to about $1.04 per cu. yd. of concrete placed on the entire job.—C. BACHMANN

Remarkable foundation jobs during construction of railroad Jungfernheide-Gartenfeld in Berlin (Germany). MAX JUSSEL. *Die Bautechnik* (Germany), January 24 and February 7, 1930, V. 8, No. 4 and 6, p. 45-8, 74-7.—Railroad crosses suburban tracks on bridge of 196.8 ft. span. Abutments show very interesting construction. Unfavorable underground conditions made very deep foundations necessary. Chemical analysis of ground water showed presence of aggressive and corroding substances. Abutments were founded on caissons, made of high early strength reinforced concrete, 1:1.5:3 mix. Concrete of normal cement in 1:2:6 mix was used for construction of base of abutments. All parts exposed to ground water were protected by wall of acidproof bricks, cemented with high quality cement mortar (1:2). Three inner walls divide each caisson into 6 cells. These walls were provided with openings. Excavated material was brought out by two elevators. Con-

crete from mixer of 13½ cu. ft. capacity was distributed by tower and dump cars. Plastic concrete was used for caisson construction. Hardened concrete was very dense. Placing of caissons took 17 days at rate of approximately 6 ft. per 24 hours.—A. E. BEITLICH

Reinforcing structures with gunite. J. F. SALMON. *Canadian Engineer*, Sept. 2, 1930, V. 59, No. 10, p. 281-283.—The Canadian National Railway has made use of gunite to protect and prolong the life of structures which would otherwise have required replacement at high cost. Disintegration of Mount Royal tunnel at Montreal resulted in high maintenance cost as well as danger to traffic due to falling rocks. Using no reinforcement a ¾-in. coating of gunite was applied to interior at a cost of $30,000, since which time all trouble has been avoided with no cost for maintenance. Brick piers in the Humber River bridge at Weston, Ont., were disintegrating and it was decided to replace them at an estimated cost of $500,000, after an earlier attempt to retard disintegration by guniting directly into the brick surface, which was not successful. The old gunite coating and all loose material was removed, joints raked out and surface sand blasted. Metal fasteners were grouted into the joints and reinforcement consisting of rods, angle irons and a 2 in. x 2 in. No. 8 wire mesh were attached with a space of 1½ in. between reinforcement and face of pier, after which gunite was applied. Cost of Weston job was $60,000. Repair of Cap Ronge viaduct, Quebec, at a cost of $70,000 saved rebuilding at a cost of $2,000,000. Concrete piers of Blende River viaduct were repaired with gunite for $65,000, thereby postponing a replacement charge of $1,000,000. All these repairs were made without interference with traffic.—G. M. WILLIAMS

Construction of the German waterway commission (Reichswasserstrassen Verwaltung) in 1929. GAEHRS. *Die Bautechnik* (Germany), January 10, 24, February 7, 21, 1930, V. 8, No. 2, 4, 6 and 8, p. 21-5, 48-52, 78-82, 114-8.—Comprehensive article describes new construction, enlargements and repairs of ship channels, locks, power stations and harbors in Germany including following concrete and reinforced concrete structures: (1) lock Anderten near Hannover, (2) highway bridge near Peine with 190.3 ft. span, (3) abutments of 278.9 ft. railroad bridge Peine-Braunschweig, crossing Mittelland channel at angle of approximately 41 degrees, (4) three-hinged reinforced concrete arch bridge near Meerdorf with 157.5 ft. span, (5) sewer systems near Aue and Allen consisting of high pressure concrete pipes with 2 openings of 40.4 sq. ft. and 3 openings 12.79 ft. square, (6) new locks in river Weser, main lock being 738.2 ft. long and 41.0 ft. wide, and (7) 7 locks in the river Neckar. Concrete used for all these structures was made from low lime portland cement, since ground water had corroding action.—A. E. BEITLICH

Use of under-water cast concrete in Sweden. F. TRIER. *Die Bautechnik* (Germany), February 21 and March 7, 1930, V. 8, No. 8 and 10, p. 109-12, 142-4.—Methods of under-water casting of concrete are described. Process can be carried out (1) with pipes, (2) with caissons, (3) by pouring and (4) by pressing cement mixtures. Best mixtures of concrete for pipes, movable in vertical direction, range between 1:2.5:3 and 1:4:5 (cement, sand, gravel). Caisson method requires mixtures from 1:2:4 to 1:3:4. Bridge over Skellefte river was built by pipe method. Concrete was poured in layers 1 ft. thick and 3 ft. wide, special care being taken to prevent concrete from sinking in pipe below water level. Concreting was continued through 3 days and 3 nights. Caisson method was used for construction of foundation of one pier of Klara bridge in Stockholm. Methods (3) and (4) can be used only when water is not very deep. Laboratory tests were made to study effect of working, consistency and proportioning of mixture and composition of aggregates. Results showed that concrete cast under water is influenced much more by above factors than ordinary concrete. Proper consistency must be maintained. Cement content

must be higher than for ordinary concrete to obtain same strength. Percentage of fine material should be slightly greater than indicated by Fuller curve. Formation of mud must be prevented by continuous pouring. Numerous construction jobs are described, illustrating applications of above methods. More detailed description is given of construction of coffer dam in Beckholmen (Sweden), and quay near Wallvik (Sweden). (cf. W. Nakonz. *Die Bautechnik* (Germany), Jan., 1930, V. 8, No. 3, p. 35-6.)—A. E. BEITLICH

Waterproofing of concrete structures. H. R. LORDLLY. *Contract Record Eng. Rev.*, April 16, 23 and 30, 1930, V. 44, No. 16-17-18.—Methods of waterproofing concrete structures has not kept pace with increased use of concrete. Experience has shown that the old idea of "a little more cement in mortar or a richer finish" does not give satisfactory results. In a review of waterproofing, four classes of structures are involved: (1) heavy structures exposed to high heads of water, such as dams; (2) lighter structures such as reservoir walls which may be waterproofed by using membrane near water side of wall or by use of admixture; (3) unit pieces of artificial stone or concrete waterproofed during manufacture with an admixture and (4) foundation walls in contact with wet soil which can be protected by tarring or asphaltic paint. Requirements for a perfect membrane waterproofing include flexibility, elasticity, toughness and tensile strength of fibre and ability to absorb and retain material used to saturate membrane fibres. Pointers on waterproofing basement floors, basement or cellar walls, concrete roof, waterproofing by admixture, surface waterproofing and surface waterproofing with impregnating solutions are included. Deductions from experience in placing sub-aqueous concrete through spout or tremie are summarized.—G. M. WILLIAMS

A new construction principle. GEORG EHLERS. *Der Bauingenieur* (Germany), Feb. 21, 1930, V. 11, No. 8, p. 125-32.—Construction of coal bin with capacity of 140,000 cu. ft. shows application of new principle in reinforced concrete construction practice. Structure is 197 ft. long and 32.8 ft. wide and consists of 8 single cells, each 31.2 ft. high. Total height of structure is 75.9 ft. It is supported by 2 single frames at the ends and 3 pairs of double frames under bins. Dimensions of columns of double frames are 2 ft. 9 inches by 4 ft. 3 inches, those of single frames are 2 ft. 6 inches by 4 ft. 3 inches. Distance between columns of double frames is 8 ft. End and double frames are placed 42.6 resp. 43.9 ft. apart. Main feature of structure is complete lack of stabilizing beams and girders. Walls 8 in. thick and bottom 10 in. thick are unsupported over about 42 ft. and only their unique connection and joint compensation of certain stresses made construction possible. Mathematical calculations on which construction was based are fully described. (cf. H. Craemer, *Zement* (Germany), July 24, 1930, V. 19, No. 30, p. 703-8.)—A. E. BEITLICH

Winter jobs in concrete and reinforced concrete construction. WILHELM PETRY. *Der Bauingenieur* (Germany), March 14, 1930, V. 11, No. 11/12, p. 175-9.—Author describes a number of winter jobs: construction of monolithic reinforced concrete stack, 262.5 ft. high, for ammonia plant in Merseburg (Germany), foundations for asphalt plant, reinforced concrete building for filtration plant, upper stories of a big warehouse in Leipzig (Germany), two new plant superstructures and enlargement of big hospital in Muenster in Westfalen (Germany). Experience, with those structures lead author to following suggestions: High alumina cement is used with advantage due to high heat of reaction which is generated during setting process. Materials must be preheated and least amount of water should be used for mixing concrete. Reinforcements and formwork must be cleaned from ice and snow. Working and placing of concrete should be made in rooms, which are protected

against frost. After placing, concrete must be covered with canvas, straw or other similar materials, especially over night, or can be directly heated by means of coke burners.—A. E. BEITLICH

Damming, blowing and sealing in crib house construction. CHAS. C. MOODY. *Concrete*, Aug., 1930, V. 37, No. 2, p. 13-16.—In building the crib house, or tunnel terminus, for the water intake on a large power house construction job in Illinois, novel method of driving 14 in. interlocking sheet piling was involved. In order that the sheet might go down straight into the ground the first pile was driven but 10 ft. Another was started and driven 8 ft., another 6 ft., and another 4 ft., and the last 2 ft. Then the hammer was started again on the first, and each one driven 2 ft. more. The steel sheeting was but 12 in. from the building at top and in its driving the piling had a tendency to go toward the building, leaving 9 in. in which to work at bottom. Depth of excavation was about 33 ft. below top of sheeting. With specially devised pipes and air hose, only a very short time was required to remove all sand and stones to the depth required. The slot, between steel sheet piling and old foundation was sealed by concrete placed through an 8 in., 16 gauge galvanized sheet tube 25 ft. long with lower end sealed. When the pipe was full, it was slowly raised to release wood seal at bottom and operation continued until concrete in slot formed a layer 4 ft. thick. When excavation in cofferdam had proceeded to approximately same elevation as seal on opposite side of sheeting, water was let out of slot by burning a hole in steel sheet just above the concrete level.—C. BACHMANN

ROADS AND PAVEMENTS

Modern European developments in road construction methods. E. NEUMANN. *Die Bautechnik* (Germany), Feb. 14, 1930, V. 8, No. 7, p. 90-4.—Development of modern road construction methods in Europe, especially in France and Italy, is described. Special consideration is given to concrete roads. —A. E. BEITLICH

Old Pavements embedded in widened highways. *Concrete*, Sept., 1930, V. 37, No. 3, p. 17.—Design worked out salvaged old 18 ft. pavement laid 8 years ago on National Old Trails road out of Indianapolis and consisted of two thickened edge slabs 10 ft. wide and the resurfaced center portion. Resurface has minimum thickness of 5 in. over old pavement and additional foot on each side is 9 in. thick. Reinforcement placed over center portion only, consisted of an assembled bar mat of ½ in. square deformed bars. Transverse bars were 20 ft. 10 in. long with ends bent around 180 deg. for anchorage. Two construction joints were provided where the side slabs join the center portion, and a longitudinal dummy joint 1½ in. deep and ¾ in. wide down center of middle section.—C. BACHMANN

Concrete road construction in Germany. "Betonstrassenbau in Deutschland." RIEPERT. 1929, 311 pp. Zementverlag, Charlottenburg (Germany), R. M. 4.80. Reviewed in *Zeitschrift des oesterreischischen Ingenieur und Architekten Vereins* (Austria), April, 1930, V. 82, No. 15-16, p. 136.—Technical development in concrete road construction during the recent years is described. Standard specifications for road construction and construction materials are discussed.—A. E. BEITLICH

Cement and bitumen for concrete road construction. WALTER OBST. *Teer und Bitumen* (Germany), July, 1930, V. 28, No. 19, p. 315-6.—Concrete which is made with an addition of tar or asphalt emulsions to mixing water is

excellent road material. These additions do not lower cementing properties or strength of cement. Blast furnace slag cement is especially suited for this purpose. Discussion is given of several American, French and Swiss patents and construction methods are described.—A. E. BEITLICH

Concrete roads in the Elbe river region near Hamburg (Germany). ADOLF WIESE. *Die Bautechnik* (Germany), Feb. 1930, V. 8, No. 7 and 14, p. 106-8, 221-3.—A number of new concrete roads are described. Article includes details of interesting construction methods.—A. E. BEITLICH

Concrete and asphalt road surfacing. R. E. CLARKE. *Cont. J.* (England), June 4, 1930, V. 102, No. 2660, p. 1819.—Variation in length of road slabs is caused by contraction during hardening amounting to ⅛ in. in 50 ft., expansion and contraction due to temperature change, causing ¼ in. in 50 ft. for 80° F. variation, and a much smaller change in length due to alternative wetting and drying. Tensile stresses which depend on length of slab, and friction between sub-base and concrete, cause cracks which may be localized by expansion joints. Such joints are weak points due to (1) weakening of bearing value of sub-base from penetration of water, (2) relative vertical displacement of concrete, (3) impact shock at uneven joints, (4) concentrated load at end of slab. These joints may be strengthened by raising bottom reinforcement to near surface, or by building independent supporting slab under ends of main slab. Longitudinal expansion joints may be used as "white lines" and studs inserted at important junctions; and transverse joints made to break joint at 50 ft. intervals. Expansion joints left proud and burnt down with hot iron tend to become damaged by traffic. Low joints grouted with Mexphalt are recommended, joint being dried with blow torch to save time, if necessary. Joint composed of cork and bitumen is now obtainable and is claimed to have power of expansion and contraction without protrusion. Goldbeck's experiments show that coefficient of friction between slab and average sub-base may reasonably be taken as 2. From this and assumed allowable tensile stress on concrete of 50 lb. per sq. in., 8-inch slab should have joints every 48 ft. Rough surface concrete has been successfully obtained by using 2 in. granite as coarse aggregate for top course. Well tamped concrete was brushed with thick brush without disturbing aggregate until laitance was removed to depth of ¼ in. It is important to see that aggregate is properly exposed or surface becomes smooth under traffic. Elsewhere matrix has been laid for top course and coarse aggregate afterwards laid on top. Heavy tamping is required and the surface is not so level. Finished surfaces were covered with waterproof canvas on frames to protect from sun rays and winds, and when safe, sprayed with water and covered with 2 in. of continually wetted sand, for four days for "ferrocrete" and 14 days for portland cement. Before traffic was allowed over, the surface was swept and washed and treated with a 25 per cent solution of silicate of soda in three applications. It is thought that hydrate of lime liberated during setting is converted into silicate of lime which is a hardener. Average output with gang of 24 men (including a man checking level of channels) was 280 sq. yd. using a Millar mixer 14 L, which gave 30 per cent over its rated capacity of 14 cu. ft. unmixed. Importance of using first class materials and watching water-cement ratio is stressed. No advantage beyond bringing to the surface a laitance of drowned cement is obtained by tamping a too wet concrete. Care should also be taken to see that sub-base is sufficiently saturated with water before depositing slab, or moisture from concrete will be absorbed and cracks formed. It is also found that cement and fine aggregate are drawn away from surface.—JOHN E. ADAMS.

Roads and road making. *Irish Builder Eng.*, (Ireland) Aug. 30, 1930, V. 72, No. 18, p. 774.—Experiments in connection with design and construction of con-

crete roads and foundations. 3 miles long, 30 ft. wide, formed on a ballast bank varying in depth from 2 to 20 ft. An 8 in. reinforced concrete slab was laid on a 4 in. consolidated bed of clinkers and the surfacing was 2 in. stone filled asphalt mat. The bays of concrete were 14 ft. 6 in. long and full width of road. At every fourth transverse joint, bituminous sheeting was inserted for expansion, and a 12 in. by 8 in. concrete beam was formed under the joint. The mix was 1:2:4 and the reinforcement 4.7 lb. per sq. yd. The road carries 10,000 tons daily and has been used for two years. No defects were observed during the first year but cracks developed later at 14 ft. 6 in. intervals or multiples, obviously where joints in the foundation occurred. The position of the cracks in the asphalt are such as to show that heavy vehicles depress the edge of slab below level of those they are approaching. It is found too, that the slabs twist upwards and downwards as the temperature varies, and that contact with subsoil may be lost at edges during some periods. Interlocking joints would appear to remedy these defects and the subject is being investigated by Ministry of Transport.—JOHN E. ADAMS

Keeping ahead of the times in paving practice. *Concrete*, August, 1930, V. 37, No. 2, p. 22-24.—Continual effort on the part of engineering department and contractors to take advantage of all betterments and to pioneer in methods is evident in quality of concrete pavement placed in Bethlehem, Pa. Crushed and washed limestone screenings or chips are used as fine aggregate. This material was adopted because of its considerably lower price and because it compared favorably with Ottawa sand. Its use has resulted in increased compressive strengths in test cylinders and pavements. Absorption tests made from time to time are closely supervised and conducted under laboratory conditions. Slab now laid is uniform 7 in. thick and feature of design is use of thickened edges along car track areas, manholes, etc. Thickened edge is not used along curb and gutters or longitudinal joints. Pavements are divided longitudinally for traffic guidance and to facilitate laying. Longitudinal joints are made with deformed plate. Dowels are spaced at 3 ft. intervals. No dowels are used in transverse joints. A deformed metal plate carries a premolded filler. Plate is pointed with hot asphalt and filler pressed on with road roller.—C. BACHMANN

Quebec bridge roadway construction. See FIELD CONSTRUCTION—Bridges.

WATER WORKS

New McTavish pumping station, Montreal, Que. C. J. DESBAILLETS. *Contract Record Eng. Rev.*, April 2, 1930, V. 44, No. 14, p. 385.—McTavish Pumping Station, rebuilt and enlarged by Montreal Water Board, together with reservoir serving high level portions of the city is of Scotch baronial style of architecture to conform with nearby buildings of McGill University. Exterior is of random ashlar limestone and reinforced concrete frame with footings in solid rock. Floor slabs are 4 in. to 6 in. thick except around conduits where 8 in. of concrete is used as filler with 1½ in. of cement mortar finish. Reservoir is excavated in solid rock but to reduce seepage, bottom is paved and side walls waterproofed. Pavement consists of 6 in. concrete slab, reinforced with "Steel crete," divided into panels 20 ft. x 30 ft. with ⅞ in. Minwax expansion joints. Pavement was carried five feet up side walls with remainder of walls coated with gunite. Pavement 20 ft. wide has been placed outside retaining walls at top to intercept surface water from higher ground.—G. M. WILLIAMS

Enlargement of water tank in Schwaebisch Gmuend (Germany). (See ENGINEERING DESIGN—Water Works.)

North Bay's new water supply system. (See ENGINEERING DESIGN—Water Works.)

Reinforced steelwork tower, Harpenden. *J. Institution Municipal County Eng.* (England), Sept. 2, 1930, V. 47, No. 5, p. 262.—Steelwork in a 350,000 gallon capacity water tower, 85 ft. high forms a rigid, pre-erected steel structure designed to carry the dead weight of the temporary centering and wet concrete. No scaffolding is required, centering being suspended from the steelwork. In this case, when steel is initially stressed by the dead load, and before the concrete sets round it, a higher working stress is permissible for watertightness. To avoid irregular cracks, the planes of stoppage in concreting were predetermined, and joints grooved in the water face and pointed with an elastic compound. No waterproofing compounds were added and tests showed efficient watertightness. The tank is carried on three circles of columns, containing 12, 12 and 6 columns respectively, all suitably braced radially and circumferentially.—JOHN E. ADAMS

SHOP MANUFACTURE

How products manufacturers may increase business volume. AUSTIN CRABBS. *Concrete*, August, 1930, V. 37, No. 2, p. 17-19.—From the writer's own business experience it is his belief that a products manufacturer may logically and profitably become a dealer within following limitations: (a) He may be a jobber (or a manufacturers' agent) for various concrete specialties not made at his plant. (b) He may be the manufacturers' agent for incombustible building specialties other than concrete products. (c) He may deal in so-called permanent building material specialties in common use but which may not be sold by other local dealers to any great extent. Products manufacturer is supposed to be equipped to make and sell materials to be bought and used by contractors in construction. If he undertakes to enter contracting business as sideline he will be competing against very contractors to whom he hopes to sell some of materials he manufactures.—C. BACHMANN

New dry paint for the concrete brick industry. BELANI. *Zement* (Germany), July, 1930, V. 19, No. 29, p. 690.—New Paint is prepared from iron ore of high purity from Kaernten (Austria) which contains up to 94 per cent Fe_2O_3 and Fe_3O_4. It is ground to fineness of 2 per cent residue on sieve with 6400 meshes per cm.^{2} Paint is absolutely weather- and fade-proof. SO_2 and H_2S have no effect and it stands temperatures up to 160° C. without darkening.—A. E. BEITLICH

MATERIALS

ADMIXTURES

Diatomite as concrete admixture material. ELLIOTT S. HASTINGS. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 55.—Representative diatomites were selected for a series of tests, all other materials being uniform. It was found that increase in compressive strength varied with alumina and iron oxide contents of the diatomites, a table showing increases from 16 per cent to 28 per cent in compressive strength with additions of 3 per cent diatomite which varied from 2.40 per cent to 17.86 per cent alumina and iron oxide. Author says he has never found a loss of strength from addition of any diatomite, all cases reported being found due to conditions of mixing and pouring. Reduction of laitance and efflorescence has been found proportional to silica present. Amorphous silica present presumably combines with lime liberated from cement thus reducing efflorescence, and absorptive power of diatomite prevents laitance. There appears to be a definite relation between lubricative effect, or workability added by diatomite, and shape of particles. Photomicrographs and chemical analysis are given for various dolomites used in the strength tests. It is considered a matter of economics whether the desired strength should be secured by the use of the proper amount and type of diatomite or regulation of the water-cement ratio.—EDMUND SHAW

AGGREGATES

Breeze and clinker concrete. *Engineering* (England), July 11, 1930, V. 130, No. 3365, p. 41.—The British Department of Scientific and Industrial Research has issued Special Report No. 15 dealing with corrosion of steel by breeze and clinker concrete. It was found, generally speaking, the corrosive power of concrete increases with increasing sulphur content of the aggregates, and that sometimes even a small amount of sulphur left in well vitrified clinker will provide rusting. The extent of this action is reduced when fines in clinker aggregate is replaced by sand. Rusting may occur under dry indoor conditions. It is recommended that safe practice requires the use of coal residue of any kind as aggregates to be forbidden in concrete if the material is to come in contact with steel when in use.—G. M. WILLIAMS

Use of sand from dunes for concrete structures in sea water. A. GUTTMANN. *Bautenschutz* (Germany), March 20, 1930, V. 1, No. 1, p. 6-10.—Fact that many failures of concrete structures in sea water occurred when sand from dunes was used, was investigated and it was found that sand from dunes has 26.1 per cent more voids than normal river sand. Strength tests were made with regular and dune sands. In order to obtain same density with dune sand, 23.6 per cent more cement was required. Dune sand is suitable for sea water structures, when used in rich mixtures. Latter can be made normal, when concrete is allowed to harden in air before placing in water.—A. E. BEITLICH

The bulking properties of microscopic particles. PAUL S. ROLLER. *Ind. Eng. Chem.*, Nov., 1930, V. 22, No. 11, p. 1206-8.—By means of an air analyzer for fine powders, an anhydrite, gypsum, portland cement, and chrome-yellow powder were separated into homogeneous fractions above 1 micron surface mean diameter. The bulkiness, or volume per unit weight, was found to increase with decrease in particle size below a critical diameter; for particle sizes greater than the critical diameter bulkiness was constant. For all powders the same functional relationship was found to hold between voids per gram and surface mean diameter. The differences in the value of the constants for the different powders appear to depend chiefly on the particle shape, whether

cubic, prismatic, basal, etc. As would be expected from a consideration of packing of small particles in voids of larger particles, surface mean diameters of portland cement and chrome-yellow microscopic powders calculated from the bulkiness of the powder were found to average several tenths of a micron higher than true mean diameters. With this correction in mind, for powders above 1 micron surface mean diameter, a rapid method presents itself of estimating the mean particle size of the microscopic powder from a knowledge of its curve of bulkiness vs. particle diameter, but no knowledge as to distribution of particle sizes in the powder is thus obtained.—ROY N. YOUNG

Flexibility. Aim of this crushed stone operation. *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 33.—New plant of the Elmhurst-Chicago Stone Co. replaces two old plants. Outstanding features are: A carefully worked out spout arrangement permitting any mixture of sizes or sending sizes to be recrushed; the use of vibrating screens for both scalping and sizing; the use of conveyors instead of bucket elevators throughout; a surge or equalizing bin and a variable speed feeder to control the flow of the stone to the screening plant; starting and stopping of all screen house equipment from one of several conveniently located push buttons; safety device in surge bin to give warning when the bin is nearly full and to stop conveyor when it is full. Only steel and concrete are used in construction. A concrete products plant is operated and the office and some other buildings are made from its product. An unusually complete machine shop is included.—EDMUND SHAW

Aboard a lake-going sand and gravel plant. *Rock Products*, Sept. 13, 1930, V. 33, No. 19, p. 41.—Sand and gravel dredge Brazil of the Marine Dredge and Gravel Co., Chicago, makes sand for fine concrete aggregate by a double classification. The sand is pumped by two 18-in. pumps, each supplying its own screening and classifying unit. Stream goes first to a large tank classifier which overflows about one-half of the solids, which is unwanted fine sand. After passing a screen to take out gravel the sand and water go to a surface current classifier with three spigots. These discharge three products, coarse, medium and fine. Any part or all of any one of these may be wasted and by mixing the required amount of each the grading may be kept constant. —EDMUND SHAW

Standardizing aggregate sizes in Los Angeles. HARRY D. JUMPER. *Rock Products*, April 12, 1930, V. 33, No. 8, p. 65.—As many as 22 sized aggregate products were formerly sold in Los Angeles, each having a different trade name. After a merger of larger companies 11 products were made standard. The aggregates used for cement concrete have been designated as "gravels" and the aggregates for bituminous pavements as "crushed rock" although both are largely crushed gravel and boulders. The sizes for concrete range from No. 1, 2½ to 1½-in. to No. 4, ½-in. to 10 mesh. Gravel No. 123 is a mixture of sizes from 2½-in. to ¼-in. This is not used as much as formerly, practice being to deliver separated sizes to job, thus avoiding segregation.—EDMUND SHAW

An independent study of aggregate characteristics. LEROY C. GILBERT AND H. F. KRIEGIE. *Rock Products*, June 21, 1930, V. 33, No. 13, p. 58.—Following their investigations on gradation and voidage these authors describe an investigation into comparative merits of gravel, stone and slag as concrete aggregates. Investigation takes in not only their own tests but those of Kellerman of the Bureau of Public Roads and of Walker and Proudley. Mixes were made for a constant mortar-voids ratio and cement content was checked by analysis. Differences in compressive and transverse strength were finally ascribed to poor gradation, the weak concrete being deficient in the ¼-in. to ¾-in. particles. This was checked by specimens which had these sizes in

excess and those having just the amount to make a smooth distribution curve. The distribution curve is a form of direct plot of sieve tests in which each sieve is represented by a vertical line. The height of a point on any line represents the percentage remaining on the sieve it represents. Points are connected to form a curve. Horizontal spaces between the vertical lines have no value. Gap gradings and excess of any size show up better than with other plots. The authors conclude that any notable differences in the concrete making quality of the aggregates tested must be ascribed to inferior gradation where they cannot be ascribed to differences in cement content or mortar-voids ratio. It is suggested that different gradations of sand might sometimes be specified for stone than for gravel, the stone (or slag) with high voids requiring a coarser sand. The most efficient combined aggregate was found to have from 21 per cent to 29 per cent of 10-mesh to ¾-in. material. Differences of 10 per cent in strength can result more easily from changed gradation than from a change from one type of aggregate to another. Until our present conception of fine aggregate as ¼-in. to 0 changes we must supply the intermediate material with fine material in the coarse aggregate.—EDMUND SHAW

The missing link in aggregates. LEROY C. GILBERT AND H. F. KRIEGE. *Rock Products*, June 21, 1930, V. 33, No. 13, p. 55.—The authors review the work of Goldbeck and Smyers on the relation of gradation to voids and carry it further in their own investigations. They show the relation to be: (1) The more nearly uniform the size of particles in an aggregate, the greater is the voidage. (2) When two aggregates of the same general shapes but of different sizes are combined in several proportions, the greatest void reduction occurs with approximately 50-50 per cent combinations. (3) The more widely the two fractions to be combined differ in size, the greater is the void reduction. Thus a mixture of approximately 50 per cent 2-in., 1½-in. aggregate with 50 per cent below 200-mesh material is the densest observed mixture of any two separates between these two extremes. (4) As the "gap" between coarse and fine aggregates is increased, voidage of each portion increases while voidage of combined portions decreases. Thus the separate portions 2 in., 1 in. and 28 to 100-mesh particles have higher voidage than 2 in., ½ in. and 8 to 200-mesh particles respectively. However, the voidage of the 2 in., 1 in. and 28 to 200-mesh sizes combined is less than that of the 2 in., ½ in. and 8 to 200-mesh sizes combined. (5) When more than two fractions are mixed lowest voidage results from combination which provided sizes of particles just fitting into the voids of each larger fraction or combination of fractions. With spheres of unit diameter the next smaller separate which can be used to just fit without crowding into the interstices is 0.414 units in diameter. Authors conclude that low density mixtures are not best aggregates on account of their "gap grading." Most complete aggregates are short in ½ in. to ¾ in. size.—EDMUND SHAW

Promotion of premium aggregates. STANLEY M. HARDS. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 70.—Opinion is expressed that engineers are beginning to discriminate among concrete aggregates, and producers will do well to investigate their products and reprocess if necessary. Adoption of trial method of designing mixes encourages the practice of discrimination. The author gives California highway practice in the design of concrete mixes. Trial mixes are made with a definite oversanding of the voids in the coarse aggregate, the cement being fixed. Weight proportions are converted by volume to give yield. With the cement and water fixed, workability can be secured by changing the sand grading, by changing the sand proportion and by changing grading and proportion. Experience shows that sand for maximum workability with minimum water ratio will vary from 30 per cent to 40 per cent of total aggregates. Sand characteristics influence water more than coarse aggregates but both should be considered together. There is little difference in amount of oversanding required with two different coarse aggregates if these

have same voids, but if two different sands are used with the same coarse aggregate the amounts required to give the same workability will vary widely. Field practice indicates need of studying grading curves on each side of division between fine and coarse aggregates or any two ranges of sizes that may be furnished for blends to insure that sizes around the division are not deficient. The sizes from ¾ in. to 10-mesh are important not only to secure strength but because they influence the mixing time. High end fraction gradings may have low voids but they need twice the mixing time of mixes with intermediate sizes. About 50 per cent of the mix should be intermediate sizes.—EDMUND SHAW

Effect of grading of gravel and sand on voids and weights. STANTON WALKER. *Nat. Sand Gravel Bul.*, Sept., 1930, V. 11, No. 9, p. 7-10.—Report of tests in Research Laboratory of National Sand and Gravel Association in 1928 on one gravel and two sands for a wide range of gradings, as preliminary studies of gravel ballast, voids in different combinations of sand and gravel and the effect of these combinations on the strength, yield and workability of concrete. Tests were on gravel separated into the following four sizes: No. 8 to No. 4, No. 4 to ⅜-in., ⅜ to ¾-in., ¾ to 1½-in., and recombined into 57 different gradings. Most of the tests were made with the three coarser sizes; the combinations were selected so that the voids and unit weights may be estimated by interpolation for any combination of the three sizes and a limited number of the four sizes. Each of the two sands was separated into three different sizes: Fine (approximately 0 to No. 48), medium (approximately No. 48 to No. 14) and coarse (approximately No. 14 to No. 4), and were recombined into 51 different gradings, arranged in such a manner that voids and unit weights may be estimated for any other combination of them. Weights per cu. ft. were determined in cylindrical measures with height approximately equal to diameter. A measure of ½ cu. ft. capacity was used for gravel tests and of ⅕ cu. ft. for sand tests. Tests were made for two methods of filling the measure—"dry and loose" and "dry and rodded." Each value is the average of two or three tests made on each grading. While report does not pretend to discuss *effect* of voids on suitability of sand and gravel for various purposes, it does contain valuable and comprehensive information on the relationship between grading of sand and gravel and its void content. Following observations from the tests are given as a guide in studies of the data: (1) For three sizes of sand or gravel, such as used in these tests, least voids were obtained for a combination of fine and coarse sizes, with intermediate size omitted. In general about 30 to 40 per cent of finer size gave least voids. (2) Combinations of two adjacent sizes, that is, fine and medium or medium and coarse showed relatively small changes in void content for different proportions. However, as in the case of fine and coarse, a combination of adjacent sizes containing about 30 to 40 per cent of the finer size had a lower void content than other combinations of two adjacent sizes. (3) General observation may be made that, for sizes used in these tests, void content of any size was reduced by addition of a finer size up to about 40 per cent or of a coarser size up to about 60 per cent. (4) There was an approximately constant difference between the voids as determined by the loose or rodded method. In case of gravel this difference was 3 to 4 per cent and in case of sand it was 5 to 7 per cent.—P. MCKIM

Effect of grading of sand and coarse aggregate on voids and weight of mixed aggregate. STANTON WALKER. *Nat. Sand Gravel Bul.*, Oct., 1930, V. 11, No. 10, p. 1-10.—These data are from hitherto unpublished tests of the Structural Materials Research Laboratory, Lewis Institute, Chicago, released for this purpose by F. R. McMillan, Director of Research of the Portland Cement Association. Complete significance of the test results cannot be considered without taking into account effect of voids on whatever finished product utilizes the mixtures of fine and coarse material. Following indications

of relationships between grading and voids seem clear from the data and should be of interest: (1) For quantities of sand greater than required to produce minimum voids the mixtures containing coarser sand produced lower void contents. (2) Coarser sands used in this investigation had lower void contents than finer ones. (3) For gravel as coarse aggregate same percentage of sand produced minimum voids, regardless of grading of sand; the minimum voids obtained with a given grading of gravel was same for different gradings of sand. (4) For granite as coarse aggregate, percentage of sand required to produce minimum voids varied as grading of sand varied; there is, however, no indication of a definite relationship. (5) Percentage of minimum voids for each combination varied with the grading of the coarse aggregate. The larger sized coarse aggregates gave the lower void contents for mixtures of sand and coarse aggregate. This relationship seems to be a function of size of particle and, probably, of size of void spaces rather than percentage of voids in coarse aggregate.—P. McKIM

Effect of moisture content on voids in sand. C. E. PROUDLEY. *Nat. Sand Gravel Bul.*, Oct., 1930, V. 11, No. 10, p. 10-12.—Free moisture in fine aggregate causes a phenomenon known as bulking, which has been the cause of considerable difficulty in undersanded mixes and over-run in cement when volumetric proportions did not take it into account. Weight proportioning has overcome much of difficulty, but where volume batching is used it is of great importance that bulking characteristics of sand be known. From study of several investigations carried out by the Structural Materials Research Laboratory of Lewis Institute, the Kansas State Highway Laboratory, and the Bureau of Public Roads, the following conclusions have been drawn: (1) Sands increase in bulk when moist. Amount of bulking depends upon percentage of moisture and method of measurement. Bulking is accompanied by increase in percentage of voids. (2) Maximum bulking is obtained with 3 to 10 per cent moisture, depending on grading of sand. Finer sands require more moisture to produce maximum bulking than do coarse sands. (3) Fine sands bulk more than coarse sands. For the usual conditions (3 to 6 per cent moisture) 20 to 45 per cent bulking may be expected depending on grading of sand and method of measurement. (4) Bulking is not a factor in measurement of sand by weight. Weight measurements, however, should take into account weight of water present at time of weighing. (5) Inundation minimizes the bulking of moist sand and gives a value approximately equal to that of the sand measured in a dry and loose condition.—P. McKIM

Proposed German standard specifications for light cast stones made from pumice. (See SHOP MANUFACTURE.)

New forming methods for small concrete buildings. (See FIELD CONSTRUCTION—Buildings.)

CEMENT

Calculation of raw mixture. A. B. HELBIG. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 919-20.—A method is given for calculating necessary amount of raw mixture to produce a certain amount of cement. Calculations are based on wet and dry processes.—A. E. BEITLICH

Tricalcium silicate. A. GUTTMANN AND F. GILLE. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 914-5.—Methods of Hansen and W. Dyckerhoff for preparation of tricalcium silicate are described; and explanation is given for the fact that R. Brill was unable to detect characteristic lines of this compound in his X-ray examinations. (cf. R. Brill, *Zement* (Germany), Aug. 21, 1930, V. 19, No. 34, p. 796 and Jaenicke, *Zement* (Germany), Nov. 21 and 28, 1929, V. 18, No. 47-48, p. 1345-7, 1388-9.)—A. E. BEITLICH

Influence of the monocalcium phosphate on the hydration of cement. KATSUZO KOYANAGI. *Kogyo Kwagaku Zasshi* (*J. Soc. of Chem. Ind.* (Japan), July, 1930, V. 33, Supplemental binding, No. 7, p. 276-7.—It was found by microscopic and physico-chemical methods that very small quantities of monocalcium phosphate retard setting of cement without considerably influencing hardening process. This phosphate has tendency to form insoluble di- or triphosphates with calcium hydroxide, which is liberated by hydration process. Hydration of calcium aluminate seems to be especially effected by above mentioned phosphate.—A. E. BEITLICH

Temperature in the mill and setting time of cements. KATSUZO KOYANAGI. *Zement* (Germany), Oct. 16, 1930, V. 19, No. 42, p. 988-9.—Reply to article written by Heiser (cf. *Zement* (Germany), July 17, 1930, V. 19, No. 29, p. 679-80.) Author describes a number of grinding tests carried out in regular cement mill and in laboratory mill. In general, cements ground in laboratory mill show slower setting properties. Second investigation deals with influence of gypsum on setting time. It was found, that gypsum loses some of its retarding properties, when its percentage in clinker exceeds a certain limit, which depends on chemical composition of clinker. Cements with high SO_3 contents become quick setting when exposed to high temperatures in mill, while cements with low SO_3 contents remain normal.—A. E. BEITLICH

Free lime in portland cement and soundness. GUSTAV HAEGERMANN. *Zement* (Germany), Oct. 16, 1930, V. 19, No. 42, p. 982-4.—Twelve different technical raw mixtures were burned in experimental furnace under same conditions at temperatures ranging from 2237° to 2687° F. Obtained materials were carefully ground (to prevent action of carbon dioxide and water of air) and tested for uncombined lime with method by Emley and for soundness with steam and cold water test. It was found that under mill conditions a good clinker can be produced at 2327° F. Based on steam tests, range of burning temperature is within 392°; based on cold water tests it is within 212° F. Steam test can give good results with clinker with up to 2 per cent uncombined lime (generally only 1 per cent), and clinker with up to 4 per cent can stand cold water test.—A. E. BEITLICH

Estimation of lime in cement. *Engineering* (England), Sept. 5, 1930, V. 130, No. 3373, p. 314.—British Department of Scientific and Industrial Research has issued Building Research, Tech. Paper No. 9, The Estimation of Free Calcium Hydroxide in Set Cement: A Calorimetric Method. The method depends upon the fact that, within small and ascertained limits, the dissociation of calcium hydroxide begins at about 350° C. and is nearly complete after ignition at 550° C. for half an hour. If therefore two parts of a sample of material containing calcium hydroxide are first heated to these temperatures for half an hour and the resulting products are then hydrated, the difference between the two heats of hydration will be a measure of the calcium hydroxide present in the sample providing no other change has been produced in the process. The aggregate error in applying the method to portland cement should not exceed .5 per cent by weight of the sample in the value of the free lime obtained.—G. M. WILLIAMS

What causes the disturbing influence of portland cement on high alumina cement? KOGAKUSCHI AND KATSUZO KOYANAGI. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 866-9.—When portland cement is mixed with high alumina cement, setting process of the latter is greatly accelerated. This fact was investigated by chemical methods and under the microscope. Setting time is lowered very much for mixtures up to 70 per cent portland cement; mixtures with more than 70 per cent portland cement approach normal setting properties. Degree of burning of clinker influences set; clinker with high free lime content causes quick setting. Great acceleration

takes place when high alumina cement is mixed with lime solutions, even 0.5 per cent CaO in solution has great effect. Reactions of high alumina cement with water and with lime water were examined under microscope. Cement reacts with water very slowly first, but more vigorously after several hours. With lime water, reaction starts almost instantaneously and becomes slower afterwards. Authors come to conclusion that hydration of monocalcium aluminate with water is reversible and slowly, while reaction between monocalcium aluminate, calcium hydroxide and water is irreversible and takes place very rapidly, which fact causes quick setting of mixtures of high alumina cement with other calcareous hydraulic materials.—A. E. BEITLICH

The degree of separation by air separators. H. MADEL. *Zement* (Germany), Oct. 9, 1930, V. 19, No. 41, p. 958-63.—Directions for valuation of air separators, which were given by P. Rosin and E. Rammler in previous article (cf. *Zement* (Germany), 1929, V. 18, No. 26-32), are discussed. Three conditions must be fulfilled by efficient air separator: (1) separator must produce a material, which has most suitable composition for any desired purpose, (2) separator must concentrate fine flour, which is present in raw material, in final product, and (3) separator must prevent undesirable oversize particles from reaching final product. Efficiency is 100 per cent only when all three conditions are completely fulfilled. A number of equations are given for a mathematical expression of above facts which enable calculation of efficiency. Same conditions are shown also in graphs. Reply by Rosin and Rammler is given.—A. E. BEITLICH

Effect of gypsum on the decomposition of tri-calcium silicate by heat. THOMAS F. MULLAN *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 62.—Work of Myers is quoted which shows that calcium tri-silicate is decomposed by temperatures lower than that of the cement kiln. In addition it was found the decomposition to be accelerated by presence of gypsum. Experiments were made with three clinkers heated with amounts of gypsum varying from 2 per cent to 12 per cent. Free lime was determined after heating by method of Lerch and Bogue. Long heating with gypsum gave almost the free lime required by decomposition of tri-calcium silicate to di-calcium silicate and free lime. In some cases there was a slight unexplained excess of free lime. Tests of heating gypsum to 1000° C. indicated that none of the free lime was contributed by the gypsum.—EDMUND SHAW

Thielekiln. A. B. HELBIG. *Zement* (Germany), Aug. 7, 1930, V. 19, No. 32, p. 745-9.—In new type of high efficiency shaft kiln described, raw mix and powdered coal are stored in two bins under which two automatic scales are installed. Electric control device operates both scales and dumps material at same time. Screw and elevator deliver mixture to second screw in which materials are mixed and wetted with water. Wet material is pressed through steel plate with many holes about ½ in. in diameter by means of heavy steel rolls. After passing plate, material breaks up in small cylinders and is distributed over whole opening of kiln. Gases which pass upper layer have temperature of 140° to 180° C. They contain 32 per cent carbon dioxide and below 1 per cent carbon monoxide and oxygen. About 12 to 16 in. below surface begins sintering zone of clinker. Carbon dioxide of raw mixture forms with carbon of fuels carbon monoxide, which combines with oxygen of air for combustion to carbon dioxide. Combustion air is preheated. Total height of kiln is 25 ft., and its output is 120 to 140 tons of clinker per day.—A. E. BEITLICH

The Italian cement, lime and gypsum industries. R. DECKERT. *Tonind. Ztg.* (Germany), 1930, V. 54, No. 76, p. 1229-30. The Italian manufacturers of cement, lime and gypsum are in a single association. They make alumina, high strength portland, standard portland and natural cements, the

last class making up about 55 per cent of total. There are 140 cement mills with 800 kilns and 20,000 employees. Firms making cement products number 2,440 with 21,746 employees. Lime, mostly hydraulic, is burned in 586 plants with 1000 kilns and 6,000 workers. Gypsum is burned in 115 plants by about 1000 workers. Annual production in millions of barrels (200 kg.) is about: natural cement 22, portland cement 17, lime 12.5 and gypsum 4.—F. O. ANDEREGG

Stirring and mixing of slurry. A. VOGT. *Zement* (Germany), Aug. 21, 1930, V. 19, No. 34, p. 809-10.—Equipment consists of cylindrical reinforced concrete tank, about 20 ft. wide and 33 ft. high, with conical bottom for mixing slurry in wet cement plants. Its capacity is about 6,700 cu. ft. Compressed air of 2 atm. pressure, which enters tank at bottom is distributed and stirs slurry. A second cylinder, open at top and bottom, diameter of which is about half of that of tank, with openings in its sides, is held in vertical position in center of tank. Upper edge of cylinder must always be below surface of slurry. Slurry is forced upwards through inner cylinder, flows over its top and through side openings and moves downwards between walls of cylinders. When it becomes necessary to add dry raw material to slurry, a cylindrical sieve will be installed between both cylinders. Material circulates 20 times per hour. Calculations show speed of slurry and power consumption.—A. E. BEITLICH

Use of powdered coal and crude oil in the rotary cement kiln. AFTON J. BLANK. *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 53.—Records are given from kilns using powdered coal and crude oil as fuel. Heat used varies from 1,212,840 B. t. u. to 1,745,264 B. t. u. Both kilns used crude oil, but with one exception the powdered coal fired kilns gave higher clinker outputs. Author concludes that wide differences in heat consumption are partly due to composition of cement but are mainly due to differences in kiln operators. Not only fuel economy but the life of the kiln lining depends upon the skill of the burner. Instances are cited where unskilled men have burned out expensive linings of high alumina brick in a short time.—EDMUND SHAW

Studies on calcium ferrites and iron cements II. SHOICHIRO NAGAI AND KATSUHIKO ASAOKA. *Kogyo Kwagaku Zasshi* (*J. Soc. of Chem. Ind.* (Japan), May, 1930, V. 33, Supplemental binding, No. 5, p. 161-4.—Pure dicalcium ferrite was prepared by extraction of free lime from product obtained by heating of mixtures of $3CaO:1Fe_2O_3$ and $5CaO:1Fe_2O_3$ at 1350° C. and 1550° C. It is stated that dicalcium ferrite is ferrite with highest lime content. Monocalcium ferrite $CaO.Fe_2O_3$ was prepared by heating a mixture of 1 CaO and 1 Fe_2O_3. Combination begins at 1000° C., proceeds very slowly up to 1150° C. and ends at 1200° C. when entire mass melts. Solubility of monocalcium ferrite in hydrochloric acids of different concentrations (0.5 to 3.0 normal) was determined. Method was derived to separate dicalcium ferrite and monocalcium ferrite from their mixtures. Method is based on the different solubilities of these compounds in hydrochloric acid.—A. E. BEITLICH

Comparison of balls and cylpebs as grinding media. K. KOYANAGI. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 58.—Although grinding capacity of balls is larger than that of cylpebs it is rapidly reduced by cement that sticks to the balls. On this account the balls show the worse result in grinding. Cylpebs probably grind less by impact and more by friction than balls so adhering cement is rubbed off. The author found more "flake" cement in ball than in cylpeb product which tended to confirm this theory. Opening a 14-ft. ball mill that had been stopped suddenly showed a rapid increase in fines up to 6 ft. after which the curve tended to flatten out. With cylpebs the relation between fineness and length was more regular, the curve being nearer to a straight line.—EDMUND SHAW

Tube mill grinding. A. B. HELBIG. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 870-3.—Author describes different degrees of fineness to which materials in cement industry must be ground. Raw materials must be so fine, that sintering in kiln takes place at lowest possible temperature. Raw mixture and burned material will gain in uniformity with increasing fineness. All particles should have approximately same size so that they reach necessary temperature in kiln at same time. Coal for shaft kilns should be ground finer than it is now in practice in order to accelerate sintering process. For rotary kilns, coal should be uniform in particle size. Its fineness depends on percentage of volatile material. Cement should be ground very fine to uniform sizes without making operation uneconomic. Tests were made to study grinding process in compartment mills and suggestions are given for economic dimensions and practical installations of air separators. (cf. Grosse, Foerderreuther and Rammler, *Zement* (Germany), Feb. 27 and March 6, 1930, V. 19, No. 9-10, p. 189-94, 214-7; *J. Am. Concrete Inst.*, May, 1930, V. 1, No. 7, Abstr. Sect. p. 92; Naske, *Tonindustrie Zeitung* (Germany), March 20, 1930, V. 54, No. 23, p. 386.)—A. E. BEITLICH

Researches on the rotary kiln in cement manufacturing. Part II. GEOFFRY MARTIN. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 60.—This part discusses methods of calculating the fuel consumed in a rotary kiln by analysis of coal, exit gases, raw mix and clinker. Three methods are given. The first is the author's own, based on the fact that fuel consumption is a function of volume of CO_2 per 100 volumes of air-free nitrogen in exit gases. Instructions are given for making a table or curve from which coal consumption may be read after CO_2 volume of exit gases has been found. Second method is a calculation made from volume percentage of nitrogen in exit gases. Third is by the Kuhl's formula which the author has found to give low results but which he thinks would give correct results with calibration of instruments and calculated constants.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part III. GEOFFRY MARTIN. *Rock Products*, Sept. 13, 1930, V. 33, No. 19, p. 57. This reviews the author's work for the British Portland Cement Association on heat losses due to formation of carbon monoxide and admission of excess air. Statistics from 2139 analyses of exit gases show best result is obtained with present conditions when exit gases show 1.1 per cent free oxygen. But in practice this figure is never maintained, the actual content fluctuating from 0 per cent to 7 per cent. Author says this indicates that burners are vigilant in watching burning and constantly correcting air supply, but that the eye is not a sufficient guide. He proposes and explains the use of an indicator by which oxygen in the exit gases may be kept between 1 per cent and 2 per cent to save from 50 to 150 tons of coal a week in operation of a 200-ft. kiln, according to the degree of prevailing mismanagement. In many cases increases of 10 per cent to 40 per cent output may be obtained. A number of tables and charts are presented, with formulas needed for kiln testing and their derivation.—EDMUND SHAW

A development in air classification. GEORGE ADAMS ROLFE. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 47.—The air classifier described is a tower through which the material falls against an ascending current of air. Outlets with catch basins allow part of the air to escape at different heights carrying a sized product with it. The velocity of the air is greatest at the bottom and least at the top. Experiments have been made on a unit handling from three to five tons per hour. Although originally designed for separating mica, the classifier may be applied to the grinding of cement and is particularly adapted to slag grinding. The author points out that the device has no moving parts and there is nothing in it that can wear out.—EDMUND SHAW

Decrease of water content of slurry in manufacture of portland cement by wet process. PETER P. BUDNIKOFF, G. W. KUKOLEW AND W. M. LESCHOJEW. *Kolloid Z.* (Germany), Sept., 1930, V. 52, No. 3, p. 341-8.—Advantages and economy of wet process are characterized. New method is described, which enables reduction of water content of slurry without decreasing fluidity. Investigations were based on normal mixtures of limestone and marl with 48 per cent water. Heat calculations and viscosity determinations of this mixture were made. Additions of certain bases, like Na_2CO_3, Na_2SiO_3 or NaOH in concentrations from 0.005 to 0.15 normal, increase viscosity. Peptonization is caused by OH-ions. Sodium silicate shows greatest effect, while sodium hydroxide absolutely destroys fluid properties. Additional studies were made with molasses solutions. A theory is derived explaining coagulation and peptonization. Decrease of water offers many advantages, less heat is consumed, grinding process is accelerated, hardness of water is reduced by additions and dimensions of equipment can be reduced. (cf. *Zement* (Germany), Jan., 1930, V. 19, No. 5, p. 96.)—A. E. BEITLICH

New device for the removal of rings in rotary kilns. *Zement* (Germany), Oct. 9, 1930, V. 19, No. 41, p. 963-4.—A new type of cement gun is described. Special features of apparatus are great mobility, ease in aiming at any desired point and low expenses. Bullet weighs about 4 ounces.—A. E. BEITLICH

Researches on the rotary kiln in cement manufacture. GEOFFREY MARTIN. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 49—This is the first of a series of 27 articles. The author says that a complete theory of the rotary kiln has been worked out. Efficiency is barely 19 per cent showing a wide scope for improvement in design and operation. With a slurry containing 40 per cent water, fuel consumption should be about 21 tons standard coal for 100 (long) tons clinker produced. Of supreme importance in fuel economy is maintenance of as high a flame temperature as possible in combustion zone. Only B. T. U.'s above 805 C. can be directly utilized for forming clinker and merely preheat the slurry. Losses of heat by radiation and convention, both inside and outside the kiln are far more important than they were thought to be.—EDMUND SHAW

Some errors in rapid methods of mix control. S. L. MEYERS. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 60.—Reasons why the acid-alkali titration, ignition loss and lime soluble in HCl from raw or calcined raw mix are not absolute criterions of what will be the lime in burned clinker, are acid-insoluble compounds in the raw mix and variations in composition of the dust carried up the stacks of kilns by the waste heat gases. A long list of acid-insoluble lime minerals is given, the commonest those of the pyroxene group, such as augite and hornblend, and some of the feldspars and garnets. There are also lime minerals only partly soluble in acid. Dust losses vary with materials in raw mix. With high and low limestones the high grade limestone powders readily after calcination and much goes out with the stack dust, thus lowering the lime in the clinker. The effect is apt to be reversed where clay is used as the low, as the clay is more easily picked up by the exit gases.—EDMUND SHAW

Influence of use of high quality aggregates on quality and costs of concrete. "Einfluss der Verwendung von Edelzuschlag auf die Guete und die Kosten von Beton." PAUL HERTEL. 1930, Charlottenburg 2 (Germany), R. M. 2.00. Reviewed in *Zement* (Germany), Sept. 18, 1930, V. 19, No. 38, p. 906.—Results of comprehensive investigation are published concerning effect of aggregate gradation on consistency, weight, strength and water permeability of concrete. Costs of aggregates and concretes are compared. Tables and graphs explain this investigation.—A. E. BEITLICH

Rapid determination of magnesia in portland cement and raw materials. AUTHUR J. POOL. *Rock Products*, Aug. 16, 1930, V. 33, No. 17, p. 80.—The method of determining magnesia by weighing as $Mg_2 P_2 O_7$ is accurate but tedious. The author prefers a volumetric method. The sample is dissolved in dilute hydrochloric and nitric acids and iron and alumina are precipitated with ammonium hydroxide. Lime is precipitated as calcium oxalate and determined by potassium permanganate titration. Magnesia is determined on the filtrate from lime by precipitation with known volume of sodium hydroxide and titrating the filtrate with hydrochloric acid for excess. Article notes several details which must be carefully attended to for a correct result. It is especially important that all the ammonia be expelled before titrating. Magnesia is calculated from the amount of sodium hydroxide required to precipitate it.—EDMUND SHAW

Small-piece testing method for strength of cement mortars IV. S. NAGAI. *Kogyo Kwagaku Zassi* (*J. Soc. of Chem. Ind.*) (Japan), March, 1930, V. 33, Supplemental binding, No. 3, p. 290-5.—New method was derived and investigated by author to test strength of cement mortars with test pieces, which are much smaller than normal sizes. Tests were made with portland cement, blast furnace slag cement and several high alumina cements and one Soliditit cement comparing obtained results with normal testing methods. It was found, that in general lower results were obtained after 28 days combination storage than after 28 days water storage, when tested by small-piece testing method. No increase in strength of high alumina cements was observed after long water storage. Author claims, that this fact is due to solution of a tricalcium aluminate which is formed during hydration of $5CaO.3Al_2O_3$ and monocalcium aluminate.—A. E. BEITLICH

Are you getting the true lime value on your raw mix analysis? ROBERT R. KING. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 60.—Where lime is determined by either gravametric or volumetric methods after fusion with alkaline carbonates an error as much as 2 per cent to 3 per cent may be introduced. Tables show that fusion with varying amounts of $Na_2 CO_3$ from 0.2 gram to 8.09 gave from 75.48 per cent to 80.33 per cent apparent $Ca CO_3$ content. This is because sodium and potassium salts formed in fusion cannot be washed out in spite of their high solubility. Remedy suggested is to use as little alkaline salt for fusion as possible. The author states that with 20 per cent insoluble in the 0.5 grams sample of raw mix only 0.1 grams of fusion mixture is necessary.—EDMUND SHAW

Raw grinding and slurry control. R. J. BINFORD. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 41.—Maximum production in a grinding plant is only accomplished by having proper size of grinding media in each mill compartment. The author gives the formula for the proper size of largest ball. Largest practical size is 5-in. dia. as with larger balls repairs on tube mill are heavy. The feed should therefore all pass a 1½-in. screen and nearly all should pass a 1-in. screen. Control of slurry covers: Grinding to the required fineness for burning and for desired quality, maintaining as low moisture content as will allow the slurry to be handled easily and quickly and maintaining a correct chemical balance of different compounds necessary in finished cement. The finer the grinding the better the results in the kiln but as fine grinding is expensive materials are ground to point where maximum demand for quality is satisfied. For economy's sake water in slurry is kept as low as is consistent with quality. At the West Penn. plant, where the author has charge of mechanical departments, economical point is 33 per cent to 35 per cent moisture.—EDMUND SHAW

Standardizing cement sieves. E. TAYLOR. *Rock Products*, June 21, 1930, V. 33, No. 13, p. 85.—The author gives his method, used by the Cebu Portland

Cement Co., Naga, Cebu, P. I., for correcting the fineness sieve test. A 200-mesh sieve is checked and calibrated by the U. S. Bureau of Standards. A large bottle of cement, fineness of which has been determined by this sieve, is kept in laboratory and each tester is required to test a sample of this cement with sieves actually used in testing the shift's run. Results are entered on a card without correction. Results are averaged by laboratory chief and compared with true fineness of the bottle cement and the necessary corrections determined for sieves in use. These corrections are applied until new corrections are ordered. System insures correct results and has the advantage that checked and standardized sieve is used only occasionally.—EDMUND SHAW

Internationalization of standards. V. M. ANZLOVAR. *Zement* (Germany), Oct. 2, 1930, V. 19, No. 40, p. 942-3.—Standard specifications for cements were revised in several countries. Holland and Italy were first to work out specifications for high alumina cements, Spain followed their example. High alumina cements must contain more than 40 per cent Al_2O_3 and less than 12 per cent Fe_2O_3; fineness must be: not more than 0.5 per cent residue on 75 mesh sieve and not more than 6 per cent residue on 175 mesh sieve; time of initial set shall not be less than 30 minutes and final set shall not be later than 4 hours. Tensile strength is tested with neat cement briquets. New concrete testing method for compressive strength is introduced. Standard 7.9 in. cubes are prepared from following mixture: 659.2 lbs. cement per 35.3 cu. ft. concrete, 30 cu. ft. broken stones (0.37 to 0.59 in.), 14.1 cu. ft. sand (0.04 to 0.06 in.) and 34.3 gallons of water per 35.3 cu. ft. concrete. Compressive strength after 1 day must be at least 3150 lb. per sq. in. and after 28 days 3270 lb. per sq. in. Similar specifications are given for high early strength cements. (cf. Platzmann, *Zement* (Germany), Jan. 2 and 16, 1930, V. 19, No. 1 and 3, p. 4-7, 49-52; *Ibid*, July 24, 1930, V. 19, No. 30, p. 701-2; *J. of the Amer. Concr. Inst.*, March, 1930, V. 1, No. 5, Abstr. Sect. p. 48; *Ibid*, Oct., 1930, V. 2, No. 2, Abstr. Sect. p. 23.)—A. E. BEITLICH

German standard specifications for portland cement, iron portland cement and blast furnace cement. *Zement* (Germany), Sept. 18 and 25, 1930, V. 19, No. 38-39, p. 890-9, 915-9.—New proposed standard specifications, open for objections until Nov. 1, 1930, differ from old specifications in following points: Specifications for high testing cements are added, limits for compressive and tensile strength are raised considerably, definition for blast furnace slag cement was changed. Specifications are divided into three parts. Part 1. Definition and properties. (1) Portland cement: loss on ignition should be less than 5 per cent, MgO less than 5 per cent and SO_3 less than 2.5 per cent (ignited base). Ratio between CaO and sum of soluble SiO_2, Al_2O_3, Fe_2O_3 and MnO should not be smaller than 1.7. Additions of foreign materials after burning process must be below 3 per cent. Not more than 35 per cent of raw materials, dried at 212° F., should be retained on 175 mesh sieve. (2) Iron portland cement: must contain at least 70 per cent portland cement and not more than 30 per cent granulated basic blast furnace slag. Ratio between 1 CaO + 1 MgO + $\frac{1}{3}$ Al_2O_3 and SiO_2 + $\frac{2}{3}$ Al_2O_3 in slag should be greater than 1. (3) Blast furnace slag cement: this cement is ground mixture of granulated blast furnace slag (having same ratio as for iron portland cement) and 15 to 69 per cent portland cement. Slag should not contain more than 5 per cent MnO. CaO content is in general below 55 per cent. Fineness for all three cements must be 2 per cent retained on 75 mesh sieve and not more than 25 per cent retained on 175 mesh sieve. Hardening should not start sooner than 1 hour after mixing cement and water. Following strength data must be reached of 1:3, cement: standard sand, mixture:

	Cement		High Testing Cement	
	Compr.	Tens.	Compr.	Tens. Str.
After 3 days water storage	....	...	3555	355.5
After 7 days water storage	2560	256	....	....
After 28 days water storage	3910	...	....	....
After 28 days combination storage	4977	427	7110	569

Water storage means: 24 hours in moist closet then in water (63 to 68° F.). Combination storage means: 24 hours in moist closet, then 6 days in water and then 21 days in air (63 to 68° F.). Part 2 discusses sampling, preliminary tests, preparation of materials for standard tests, determination of fineness, initial and final set and soundness, also mixing, making, storage and breaking of cubes and briquets. Specimens for compressive strength tests are 2¾-in. cubes and for tensile strength are briquets with smallest cross section of 0.77 sq. in. Part 3 deals with standard sand, its chemical composition and grain sizes. Standard specifications for sieves are given. Apparatus for determining setting time, for mixing mortars, hammer apparatus and tensile strength tester are described. Exact dimensions of test pieces and their molds are given and also allowable tolerances for manufacture and wear.—A. E. BEITLICH

California's largest cement mill keeps up-to-date *Rock Products*, June 21, 1930, V. 33, No. 13, p. 41.—Oil fuel for the Santa Cruz Portland Cement Co.'s plant at Davenport, Calif., is pumped from tankers to storage tanks on shore about three miles from the plant. The line rests on the ocean bottom for one-half mile. This is one of the few places where oil is pumped from open sea to plant storage. Limestone is mined from glory holes above bulldozing chambers connected by chutes to two main haulage tunnels. Clay overburden supplies most of clay needed for the mix. Mining is safer than quarrying. Costs of opening one glory hole with bulldozing chamber and tunnels was $26,000, about 86¢ per ton for development. Blending begins by keeping coarse and fine limestone separate and combining as needed. The fines carry much clay. A second blend is made by drawing proportionately from "hi-low" bins containing crushed product. This blend goes to 40 steel hoppers and a mix from these goes to Hardinge raw grind mills with air separators, either Hardinge or Sturtevant. A heavy circulating load is carried. Grinding is 90 per cent minus 200-mesh. Hardinge mills finish grind clinker after it has been crushed in rolls in closed circuit with vibrating screens. Gypsum is calcined in Raymond "calcine-while-you-grind" mills. Six ball and six tube mills grind "oil well" cement which is ground finer and harder burned and approaches high early strength cement in its properties.—EDMUND SHAW

Modern cement testing laboratory. *Rock Products*, June 7, 1930, V. 33, No. 12, p. 75.—The new laboratory of the Southwestern Portland Cement Co., at El Paso, Texas, has been successfully designed to eliminate unnecessary noise. All grinding machines have texrope drives and are mounted on lead cushions. The rotap screening machine is placed in a cabinet lined with celotex and mounted on a lead cushion. The moist room is kept within 1° F. of 70°. It has double concrete walls insulated with tarred cork-board. There is one large door for entrance and two small doors for attending to specimens while they are setting up. A 5-h. p. Kelvinator and four Chromolox strip heaters control the temperature. They are regulated by two thermostatic mercoid switches. Sprays keep the air at almost 100 per cent humidity. A wet and dry bulb recording thermometer keeps a continuous record of temperature and relative humidity. Average range of temperature for 24 hours for practically all records is within 0.2°. Physical testing bench tops are of ¼-in. non-rusting steel plate. The bench for titrations has a glass plate with an electric light below.—EDMUND SHAW

Plant of Republic Portland Cement Co. in second year of production. *Pit Quarry*, Aug. 13, 1930, V. 20, No. 10, p. 17-34.—Wet-process plant with rated capacity of 3,600 bbl. per day has actually produced as high as 3,900 bbl. Raw material is quarried from argillaceous limestone or chalk which varies from soft earthy to fully solidified rock. A large bed of natural cement rock constitutes about 80 per cent of deposit. Broken stone from quarry is loaded by electric shovel into 8-cu. yd., side-dump, standard-gauge steel cars and hauled to plant by gasoline locomotive. Crusher will take feed as large as

2-cu. yd. and reduce it to ¾-in. in a single operation. One building houses the raw and finish grinding mills and the power equipment. The two types of limestone are fed, from the bins to two compeb mills. Slurry is ground to a fineness of 90 to 95 per cent through a 200-mesh sieve and is discharged into a horizontal sump and stirred by a double-ribbon conveyor. With a water content of 42 per cent it is handled by two 6-in. pumps, through a 10-in. line in a tunnel underneath the storage, to slurry tanks on opposite side. Same tunnel is used by pedestrians and for air, water, and electric lines. The six blending tanks with four correction tanks hold slurry for 48-hour run. Each tank has a mechanical and air agitator driven by motors through speed reducers. The chemist samples slurry in each blending tank while filling, and corrections are made by transfers. Two kilns are fired with natural gas at 1½ lb. pressure, and equipped with temperature recorders. Slurry enters the kilns through ferris-wheel feeders. Feed end of each kiln is within a dust chamber which connects by steel breeching with its 200-ft. reinforced-concrete stack. Clinker from kilns passes to coolers which discharge to the clinker storage pit. The clinker is taken by crane to finish-mill feed bins. Table feeders pass clinker and gypsum to compeb mills. A dust-collecting system on a mezzanine floor serves also as a ventilator. Pneumatic pumps in basement propel finished cement to tops of storage silos having total capacity of 153,000 bbl. Packhouse is a 4-story building adjacent to the cement storage. Cross screw extends outside to a loading track for loading bulk shipments through a 6-in. rubber hose. An air-jet in the hose keeps the line clear.—A. J. HOSKIN

Reactions during setting and hardening process of cement. (See PROPERTIES OF CONCRETE.)

Temperature and cement. (See PROPERTIES OF CONCRETE.)

PROPERTIES OF CONCRETE

Reactions during setting and hardening process of cement. TUTOMU MAEDA. *Zement* (Germany), Sept. 4, 1930, V. 19, No. 36, p. 842-3.—A theory is derived illustrating reactions which take place during setting and hardening periods and explaining hardening properties of cement. Certain constituents of cement form when mixed with water unstable compounds in a system containing one liquid phase. Second step is formation of oversaturated solution of stable compound. When a new solid phase precipitates from this oversaturated solution, it absorbs certain constituents of the liquid phase, which form a film around particles. Conditions for a proper hardening are: (1) particles, which separate from liquid phase, must be small, which gives great surface, (2) particles must be separated by adsorbed film, thickness of which lies in molecular range. No liquid phase should be present. (3) Attractions between the different particles and particles and film must be great, and (4) particles must have great strength. A number of experiments with ultra-microscope, which seem to prove existence of colloidal parts, are described. (cf. *Supplement to Scientific Papers of Inst. of Phys. and Chem. Research* (Japan) Tokyo 1928, V. 8.)—A. E. BEITLICH

Temperature and cement. OSCAR FABER. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 566-569.—Curves are given showing relation between temperature and time of setting for different kinds of cement. A case is cited where a recorded rise in temperature of 50 degrees occurred in three days after placing of the cement. Use of this property of heat production can be made by insulating concrete thus preventing concrete from cooling too rapidly until it is hard. Reference is made to

Bulletin No. 81, Engineering Experiment Station, University of Illinois, to point out the increase in the strength of concrete with increase in the temperature at which it is cured.—JOSEPH MARIN

Chemical characteristics of cement pipe lining. E. L. CHAPPELL. *Ind. Eng. Chem.*, Nov., 1930, V. 22, No. 11, p. 1203-6.—A brief review of literature regarding use of cement pipe lining and theories advanced to explain its effectiveness as a protective layer are given. Data are presented showing change in chemical composition that took place in several types of cement during exposure. The ages of the linings examined varied from one to two years with hot water exposure and from five to 60 years with cold water exposure. It was found that calcium is gradually leached out and iron hydroxide deposited and that this exchange did not result in disintegration of lining nor alter the effectiveness of the coating in preventing corrosion. The deposition of iron hydroxide will take place in a layer of sand and finally cement the sand into a hard coating. Cement pipe linings have been found to be excellent protection in tuberculating water and also good results were obtained with salt water and sulfur water. If CO_2 content in steam is high in steam return lines, it has a solvent action on both the iron and calcium compounds and destroys some cement linings.—ROY N. YOUNG

The elasticity of concrete. H. H. LANGDON. *Monthly Bull. No. 30*, Washington State College, April, 1930, V. 12, No. 5, 48 p.—Data were taken from three sources: student work, control specimens from campus construction jobs, and from field jobs. Special apparatus used to measure deformations, consists of two steel rings, each clamped to the specimen by three equally spaced screws. Connected between the rings are three equally spaced metallic bellows. The three bellows are interconnected by means of small tubes. Another small tube extends from one of the bellows to a glass tube placed at a small angle from the horizontal. Any deformation in the specimen forces a small quantity of water through the bellows and tubes into glass gage where it is easily read. A comparison was made between the results obtained with this type gage and with mirror type. Satisfactory agreement was found. A screw machine and hydraulic machine were used to produce loads. Two hundred and forty specimens were tested. Tests were conducted to prove that slight curve at lower end of stress-deformation diagram was due to a lack of accuracy in the weighing of loads, or in part due to the compressometer or to temperature changes, and is not indicative of true deformation within the concrete. The major conclusions state that: concrete is elastic in the same sense that steel is considered elastic; elastic limit occurs at approximately 37 per cent of ultimate; the relationship between elastic limit and ultimate stress is fairly constant; if elastic limit is not exceeded, concrete may be reloaded and same stress-deformation relations will be obtained; modulus of elasticity as found is considerably higher than that used in present day design; there is a relationship between ultimate strength and modulus of elasticity. There are also enumerated several conclusions that need additional verification. A resume of studies made by other experimentors in the same field is also given.—IRA L. COLLIER

Corrosion of structural steel and steel reinforcing rods encased in concrete. E. A. CROSS. *Engineering* (England), July 4, 1930, V. 130, No. 3364, p. 30.—Structural steel encased in concrete and steel rods in a reinforced concrete slab supported by the structural steel beams were noted in a 26-year old engine room of a Toronto store. The construction consisted of 18-in. steel beams, span 25 ft., spaced 3 ft. 9 in. on centers embedded in concrete which supported a 6-in. concrete slab with ½-in. diameter bars, 6 in. on centers, placed in the lower part of the slab. This floor supported an ammonia refrigeration brine tank which had been leaking for a long period. A pump room immediately below this floor contained steam mains which leaked and resulted

in condensation which kept the concrete near openings wet and at a temperature of near 100° F. Cracks were found running longitudinally with steel beams and the slab reinforcing rods. It was concluded that corrosion had been caused by the following factors: (a) Cracks in concrete slab due to omission of negative reinforcement over steel beams, (b) presence of water and brine solution on top of slab finding its way through these cracks, (c) excessive condensation at under side of slab at openings in floor combined with high temperature. In repairing, steel beams were left in position but their assistance was ignored and new concrete beams as well as new slab were poured around and above them. Mixture used was 1:2:3, giving strengths of 1500 and 1850 lb. sq. in. at 7 days. —G. M. WILLIAMS

Lightweight concretes. *Builder* (England), Sept. 12, 1930, V. 139, No. 4571, p. 436.—With modern framed structures the weight of filling walls needs be reduced to a minimum to permit use of lighter steel sections. Clinker concrete slabs are very light and fire-resistant, so long as no combustible matter is present in the clinker. Peeling of plaster from clinker concrete slab walls may be due to expansion of combustible matter in concrete. Clinker concrete should never be used to cover steel nor be reinforced, on account of its porosity and probable presence of sulphur in clinker. Slabs may be covered with plaster, or, if made with ⅜-in. crushed clinker and "fines," a smooth face may be obtained. Clinker concrete walls offer considerable resistance to penetration of moisture, notwithstanding their great permeability. Pumice concrete is another lightweight concrete, suitable for building, where its high cost is not prohibitive. While having the same advantages as clinker concrete, pumice concrete has no dangers from sulphur presence. The usual mix is six parts of pumice to one of cement, fines often being omitted, to give greater voids. Voids may be artificially produced by mixing a chemical with cement, causing it to bubble and become honeycombed with air pockets. Finely ground aluminium and zinc used in these patented processes, react with lime to form hydrogen. Considerable insulating properties against heat and sound are produced in these "cellular" concretes. Sawdust concrete is a further type of lightweight concrete, the inflammable nature of the aggregate and considerable amount of its expansion, when wet, being most serious difficulties to overcome. Addition of calcium chloride to gauging water may obviate these difficulties, producing a "half-weight" concrete. By mixing small particles of ice with cement and sand, and moving the molds containing mix to a warm room, light cellular concrete may be obtained. The ice melts and hydrates the cement which forms a thin wall round the spaces occupied by the ice. Column shuttering may be dispensed with if hollow precast blocks of lightweight concrete are placed one on the other and filled with concrete and reinforcing steel. Fire resistance is thus considerably increased.—JOHN E. ADAMS

Deterioration of structures in sea water. *Can. Eng.*, Oct. 28, 1930, V. 59, No. 18, p. 593.—Tenth interim report of Institute of Civil Engineers, Great Britain, refers to experiments on the measure of resistance of reinforced concrete to deterioration by sea air and sea water. Three hundred reinforced concrete test pieces are being prepared for exposure in artificial sea water, in a sea water tank at Sheerness Dockyard and in the sea at Takoradi on the Gold Coast. The variables include 4 different kinds of cement with trass replacing a part of the cement or part of the aggregate. Three proportions are used and the medium richness will include wet and dry consistencies. Reinforced concrete test blocks were examined at Brisbane after 11 years exposure. Tar coatings have retarded but have not prevented deterioration of embedded steel.—G. M. WILLIAMS

High-early-strength concrete. S. RORDAM. *Rock Products*, Aug. 30, 1930, V. 33, No. 18, p. 45.—High-early-strength concrete may be made by using special quick-hardening cements, or from ordinary portland cement by

control of the water-cement ratio. The author believes that latter method will give desired strength in specified time for least money and that it offers better insurance against unexpected stresses at later periods. Results of a comparative experiment made under uniform conditions are given in tables and diagrams. Water-cement ratio of portland cement concrete was controlled to give same strength as that of high-early strength concrete at three days. Considerably higher strengths at 28 days and 90 days than that of the high-early-strength concrete were obtained.—Edmund Shaw

Control of the manufacture of concrete. Luigi Santarello. *L'Industria Italiana Del Cemento* (Italy), Feb., 1930, p. 16-19.—Italian concrete industry fails to apply fully available information in regard to concrete quality control as a good deal is left to the judgment of the laborer at the mixer. Quality of concrete depends on quality of materials and how they are proportioned. Cement is accepted as of good strength. It is suggested that aggregates be subjected to petrographical analysis to detect decomposable material. Feldspar is cited as such an objectionable material not indicated by usual analysis. Combination of aggregates on basis of laws of Fuller or Bolomey is recommended to give economy and density. Water-cement ratio is stressed as the one most important factor to control. Proper apparatus is available for measuring materials for concrete, and should be used.—C. G. Clair and M. N. Clair

Exposed surfaces of concrete and their protection. A. B. MacMillan. *J. Boston Soc. C. E.*, Oct., 1930, p. 467-473.—The causes of the disintegration of exposed concrete surfaces are discussed and water is indicated as the primary factor. Water weakens concrete through solution of hydrated cement compounds, expansion or freezing and the corrosion of the steel. It gains access to the concrete mass through cracks and porosity of the concrete. The precautions necessary to delay disintegration are to use proper materials, proportions, mixings, placing and curing procedure in the original construction. In addition integral water proofing material may be applied but author favors sealing concrete surface with an elastic varnish and covering varnish by a paint. Detail specifications and procedure for repairing the surface of a reinforced concrete building are given. All rusted reinforcing steel was cleaned and painted with asphalt paint. Loose concrete was removed and replaced by 1:2 mortar mixed five hours before use. The surfaces were painted with two coats of concrete paint. The total cost was $6,048 or 12¢ per sq. ft. of gross surface area. This gives a cost of 1¢ per sq. ft. per year cost to owners for surface deterioration.—Miles N. Clair

Ready mixed concrete. Miles N. Clair. *J. Boston Soc. C. E.*, Oct., 1930, p. 474-480.—The lack of durability of concrete structures in the past is due to failure to appreciate value of definite control of the proportioning, placing and curing of the concrete. Control of concrete quality is too complex a matter to be left in hands of usual concrete foreman, and an engineer of special training is required. Small projects cannot stand expense of such control. Ready mixed concrete operations are large enough to warrant expense for control of quality. The several systems for producing ready mixed concrete and equipment used are described and their limitations discussed.—Miles N. Clair

Reinforced concrete chimney stacks. *Builder*, Oct. 17, 1930, V. 139, No. 4576, p. 664.—In reply to request for information concerning suitable aggregate for a reinforced concrete chimney stack, 150 ft. high, 7 ft. 3 in. to 9 ft. 9 in. diameter internally, with temperature of flue gases at base 750° F. and a firebrick lining to be carried up 25 ft., Building Research Station states that deterioration would be due to effect of heat on aggregate and effect of gases on concrete and cement, and stresses due to temperature gradient through the walls. Ordinary clean gravel or crushed stone should withstand the heat since

the temperature would be less than that at which calcium carbonate decomposes. Portland cement, however, would commence to dehydrate slowly and lose strength mainly on inner surface, near base. Effect of sulphur gases in fumes would only cause surface reactions, provided the concrete is dense and well compacted. Stresses due to unequal temperature and drying conditions would probably cause shrinkage cracks.—JOHN E. ADAMS

Water permeability of cement pipes. E. SUENSON. *First International Congress for Conc. Rein. Conc.*, Liege (Belgium), Sept. 1930; Publication No. VII-2, 19 pp. Editor: La Technique des Travaux, Liege.—Test pipes 3½ in. long, with 2 in. outside diameter, and walls ⅜ in. thick were made of ordinary portland cement and sand of 0.2 in. particle size in mixtures ranging from 1:2 to 1:5. Corresponding water additions were 9.2 to 8.0 per cent. Pipes remained in moist closet for 24 hours, then half of samples were stored in air saturated with water, the other half under water for 27 days. The 28 day old test pipes were then kept in air until used for tests. Volume of pipes was determined by immersion in mercury and weighing overflowing mercury. It was found that specific gravity decreases with decreasing cement content. Specific gravity is greater after air storage than after water storage. Experiments to determine water permeability were made with simple apparatus with pressures ranging up to 8 ft. water pressure, kept constant throughout tests. Theory concerning flow of water through concrete was derived. Formulas and graphs show changes of flow with increasing pressure, relation between velocity of flow and internal pressure in voids and velocity and pressure in cylindric walls. Amount of water in grams which passed walls of pipes per minute was determined under different pressures and conditions. Permeability is lowered considerably with increasing cement content. Impermeability of pipes is several hundred per cent greater when specimens are stored in water than in air storage. Storage conditions are of much greater importance than composition of mix. Lean mixtures can be made dense by storage under water. Admixtures like water-glass or soap solutions have only minor influence on water impermeability. When water is slightly acid cement will be dissolved, while ordinary water and distilled water make concrete more dense. This is partly due to swelling of cement particles and partly to infiltration of impurities. Bacteria in water have also improving effect on concrete pipes even when they are made from lean mixtures such as 1:10.—A. E. BEITLICH

ENGINEERING DESIGN

BRIDGES

Test of a reinforced concrete bridge. M. LANOS. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation in *Concrete Constr. Engr.* (England), Oct., 1930, V. 25, No. 10, p. 569-571.—Tests were made to determine effect of impact on a bridge carrying Rue Lafayette over lines of the Compagnie de l'Est in Paris. Deflections were measured for various members of the bridge. Readings were taken with loads passing over bridge at different speeds. The general conclusion drawn from the tests showed that, in bridges of this type, speed and shock do not sensibly increase stresses in principal beams. On the other hand, they produce stresses greater than those calculated in the cross girders, stringers, and the slab below the roadway.—JOSEPH MARIN

Design of road bridges. C. S. CHETTOE. *Rds. Rd. Constr.* (England), Oct. 1, 1930, V. 8, No. 94, p. 332.—Surface finish: If concrete bridges are faced with brick or masonry, dowels are not necessary, provided facing is suitably toothed into concrete backing. In exposed concrete face work, finish may be produced by use of colored cement, or special aggregates. Such facing is usually

only a few inches thick, and separated from the main body of concrete by a metal shutter, lifted as concreting operations proceed. Both kinds of concrete set together and become monolithic. Use of colored cement blocks avoids risk of patchiness. When special aggregate is used, it is usually crushed local stone. The surface skin of cement has to be removed, by some suitable method such as bush hammering or scrubbing with a wire brush or use of chemicals. The molds may also be coated with special retarding solutions, so that the surface easily brushes off with scrubbing. Further effects may be obtained by polishing with carborundum, or painting with waterproof paint.—JOHN E. ADAMS

Reinforced concrete canal bridges in Belgium. M. BOUCAU. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation in *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 561-563.—To conform with clearance necessary for the passage of barges and with topography of the land, the depth of construction available was only 2 ft. 10 in. in one case. A concrete arch bridge would not permit sufficient clearance and a steel bridge would be too costly. A satisfactory solution was found by constructing a portal with hinges at low level.—JOSEPH MARIN

BUILDINGS

High buildings of steel or concrete? K. SCHAECHTERIE. *Teknisk Tidskrift* (Sweden), Sept., 1930, No. 9, p. 124-126.—An office-warehouse building, five stories high and with a live load of 51½ and 206 lb. per sq. ft. for the office and the warehouse respectively was figured for cost and a comparison made between cost of a concrete and of a steel structure. The concrete building was found to have 1 per cent less effective floor area and to weigh 25 per cent more than the steel building. The cost for the former was 3.25 Mark against 3.73 Mark per sq. ft. effective floor area for the steel building, a difference of 15 per cent.—O. Albert

Design of wind bracing. EDWARD SMULSKI. *J. Boston Society Civil Eng.*, Nov., 1930, p. 491-523.—Details of the design of wind bracing are presented for tall buildings both of reinforced concrete and of steel, with particular attention to design of the connections of the wind bracing for steel structures. Reinforcement is required in concrete wind braces at the column to resist both positive and negative bending moments, the zone of negative bending moment extends further from the column than for an ordinary restrained beam and diagonal tension reinforcement may be required for the entire length of the beam to take care of the shear.—MILES N. CLAIR

DAMS

Unique cutoff construction and arched foundation features of Rodriguez dam. (See FIELD CONSTRUCTION—Dams.)

MISCELLANEOUS

Construction of quay at Rotterdam. W. F. VAN DIJCK. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation, *Concrete Const. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 557-61.—The quay walls will be 8500 ft. long and are designed to allow for an ultimate depth of water of 39 ft. Vertical walls were adopted to permit simplification in placing concrete, and though greater amounts of steel and concrete will be required than by using battered walls, the ultimate unit cost will be less. Details are given of method of construction. The layout of the yard and method of constructing caissons are points of interest. Coulomb's

method of calculating earth pressure was adopted, using values of $\phi = 30$ degrees and $\phi = 40$ degrees for sand below and above water level respectively. —JOSEPH MARIN

New railway roadbed construction of reinforced concrete. *Zement* (Germany), Sept. 18, 1930, V. 19, No. 38, p. 904-5.—A new type of foundation for railroad tracks is described, which can be used with great advantage for railroad bridges where vibrations of structure must be reduced to minimum. Tracks are connected to concrete foundations by means of heavy steel springs. Experiments undertaken on main line of Austrian railroad gave excellent results. (cf. Wirth, *Organ f. d. Fortschritte d. Eisenbahnwesens* (Austria), Nov. 1929, No. 21.)—A. E. BEITLICH

Concrete retaining walls. E. W. ANDREWS. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liège, 1930. Abstracted from *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 572-573.—Cross sections and relative economies of plain and reinforced concrete walls designed to fulfill as nearly as possible the same conditions for three different forms of construction and two different theories of earth pressure are discussed. The three forms of construction are: (1) mass concrete, (2) "slightly" reinforced concrete, and (3) "conventional" reinforced concrete.—JOSEPH MARIN

Examples for calculation of reinforced concrete structures. "Exemples de calculs de constructions en beton arme." LEON COSYN. Editor: Librairie Politechnique Ch. Béranger, Paris and Liège (Belgium), 1928, 3rd edition, Fr. 60.00. Reviewed in *Zement* (Germany), Sept. 4, 1930, V. 19, No. 36, p. 858.—Book is continuation of: "Practical treatise of reinforced concrete structures" (Traité pratique des constructions en béton armé) by same author. Fundamental principles of calculations, which arise in all fields of reinforced concrete construction are discussed and explained with numerous examples. In its seven parts, book deals with scaffolds; prismatic structural parts exposed to compression, tension, flexure or torsion; discussion of total structures, and gives tables for use by designer and contractor.—A. E. BEITLICH

Parabolic sheds in reinforced concrete. L. BAES AND J. VERDEYEN. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liège, 1930. Abstracted from translation in *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 563-565.—These sheds are used for storage of chemicals of a highly cohesive nature. Products of this kind cannot be stored in ordinary silos. Experience has proved that best cross section is parabolic or nearly so. A three hinged arch was used. Economical ratios between rise and span are given and dimensions and details of construction are outlined.—JOSEPH MARIN

Shaping of incomplete reinforced concrete hinges. ALEX JESINGHAUS AND OTTO BIELIGK. *Zement* (Germany), Sept. 4 and 11, 1930, V. 19, No. 36-37, p. 850-5, 873-9.—Critical study was made investigating influence of shaping of joints and placing of steel in reinforced concrete hinges. Mathematical calculations illustrate occurance of stresses in hinges of different shapes and with different reinforcements. Certain types are discussed and their application and advantages characterized. Great care must be taken in placing steel reinforcements, number of which should not exceed the necessary limit, so that uniform concreting can be done.—A. E. BEITLICH

Oregon highway bridge to be built by Freyssinet arch method. *Eng. News Record*, Aug. 21, 1930, V. 105, No. 8, p. 290.—Utilization of Freyssinet system of concrete arch construction will be feature of major interest in building Rogue River highway bridge now under construction for Oregon State Highway Commission. Seven pairs of arch ribs each having 222-ft. span and

rise of 47 ft. make up main portion of structure. At either end will be 144 ft. of concrete viaduct approach composed of 16-ft. arches, making a total length of 1,938 ft. Abutments at ends of main arch system will rest on rock and those for six intermediate piers, including two abutment piers, will rest on foundations of piles driven in gravel. Intermediate abutment piers divide arch system into a three-span group and two two-span groups on either side. The piers are designed as yielding or elastic, believed to be an innovation in use of Freyssinet method. In this method rib-shortening and shrinkage stresses are reduced by leaving an opening at crown of each rib in the concrete and inserting hydraulic jacks to allow for adjustments. These jacks permit the position of the arch to be maintained or corrected, following the construction period, until sufficient time has elapsed for rib-shortening to take place. Because of unusual design, the structure is to be a cooperative research project with U. S. Bureau of Public Roads and State of Oregon as participants.—D. E. LARSON

Design of road bridges. C. S. CHETTOE. *Rds. and Rd. Constr.* (England), August 1, 1930, V. 8, No. 92, p. 257; No. 93, p. 297.—Retaining walls: Mass concrete walls, rarely employed for greater heights than 10 ft., should be battered slightly to care for settlement under the toe. Rankine's Theory is still mainly used, and is applied to stability against sliding and overturning. Respective factors of safety should be 1.5 and 2.0. For live load, a 3 ft. surcharge is usually satisfactory to cover effect of Ministry of Transport train. All walls should be backed with dry fill, and provided with weepholes. It is essential that concrete in the toe of wall be formed against virgin soil, without fill, and that no open drain be constructed in front of wall. Otherwise passive pressure will be destroyed and sliding result. For reinforced concrete walls up to 12 ft. high, L-walls without counterforts, consisting of vertical slab, toe and heel are common. The resistance to sliding may be increased by sloping the footing slab. If the toe is much greater than the heel, to avoid excessive cutting into the bank, the stability factor is lessened, but by making the footing slab butt against the road slab, good resistance against sliding is provided. For heights greater than 10 to 12 ft. counterforts with horizontally spanning slabs are more usual. Positive moments are taken as $\frac{pl^2}{24}$ and negative moments over the counterforts as $\frac{pl^2}{12}$. Expansion joints should be provided at 60 to 80 ft. centres. Spandrel walls should be provided with sufficient joints to avoid restraint of freedom of movement of arch. Details of suitable reinforcement are also given. Criterion of allowable foundation pressure is maximum settlement allowable, and difference of settlement between one abutment and another. For any appreciable settlement a three-hinged arch or a free span should be used. For weak ground, caissons, i. e., steel or concrete vertical cylinders with cutting edges may be used. Skin friction may be allowed for, when a good foundation does not exist at reasonable depth. Timber piles are only to be recommended when under water for their entire length. The Engineering News Formula, is quoted. Details of typical reinforced concrete piles are given, and the Dutch Formula for driving is quoted. General descriptions of the best pile group arrangements in various types of bridges are described.—JOHN E. ADAMS

WATER WORKS

Reservoir at Barming, Maidstone. *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 543-549.—The reservoir is an extension of one already existing and is being constructed as close as possible to the existing structure. It is designed to withstand the stress due to either structure being

full of water. Details are given of the design and method of construction of retaining wall, floor and roof.—JOSEPH MARIN

ARCHITECTURAL DESIGN

Colored architectural concrete in large country residence. JOHN K. BRANNER. *Concrete*, Oct., 1930, V. 37, No. 4, p. 23.—Dutch Colonial type of architecture and use of colored concrete in exterior of building and on grounds are features of Woodside, California, residence built for cement company head. House is on a hill, and between two wings there is a court facing a formal walled garden containing orange trees. Reinforced concrete gable walls rise above the roof, and are ornamented with scrolls of Dutch character. Floors are of structural concrete, laid on light fabricated steel joists and surfaced with oak and teakwood nailed to furring strips in the concrete. Roof is covered with dark grayish brown shingle tile. Colored concrete is used in floor of the enclosed court, laid out in checkerboard design in black and white. Balcony above court is supported by white concrete columns.—C. BACHMANN

Concrete plastics in Afghanistan. DAHL. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 880-2.—Preparation of ornamental plastics for new City of Dar-ul-aman in Afghanistan was very difficult because of lack of suitable stone materials. Quarries of European type are practically unknown in this country. Stones are seldom longer than 24 in. or wider than 6 in. and cannot be used for greater sculptures. Faced by these conditions, architect and artist Karl Maas undertook difficult task of producing desired plastics of concrete. Clay and gypsum for molds were difficult to obtain and the latter had to be burned in small wood-fired furnace. Cement was furnished by new cement plant in Kabul. Aggregates were carefully washed and prepared. Among the many plastics may be mentioned a fountain, ornaments and pillars of royal castle in Kabul. Plaster technique is well known in Afghanistan where kind of clay-stucco is used. Very old samples of their primitive art are still preserved.—A. E. BEITLICH

FIELD CONSTRUCTION

BRIDGES

Mid-Hudson bridge paved in alternate slabs to equalize cable loads. *Eng. News Record*, Oct. 9, 1930, V. 105, No. 15, p. 581-582.—Dead load stresses on the suspension cables supporting the Mid-Hudson bridge at Poughkeepsie, N. Y., were equalized during laying of concrete by carefully planned sequence of field operations. Concrete roadway across the 3,000-ft. structure is 30 ft. wide between curbs and 8 in. thick, heavily reinforced with built-up mats of ½-in. steel rods. This pavement, together with two 4½-ft. concrete sidewalks 3 in. thick, imposes a dead load of 4,000 lb. per linear ft. on the bridge. Concrete was laid in three strips, the center one being 9 ft. 4 in. wide and the two side strips each 11 ft. 1 in. wide. An industrial railway was used to deliver mixed concrete and to handle forms and steel. The center strip was first completed by working from center toward both ends of bridge simultaneously. The narrow-gage track was then shifted to the finished slab and paving again started from middle. In this case, however, alternate panels of the two outside roadways were poured, first on one side of center strip and then on other, leaving staggered gaps. These openings were then filled alternately so that during the whole paving operation weight of roadway was distributed uniformly between cables.—D. E. LARSON

The "Albert Louppe" bridge of reinforced concrete. A. COYNE, E. FREYSSINET. *Genie Civil* (France), Oct. 4, 1930, V. 97, No. 14, p. 317-334.—The bridge, recently completed, consists of three reinforced concrete arches of 566.13 ft. net opening each, crossing the Elorn some four miles out of Brest. The very long spans are the feature of this structure and were adopted, after much debating, to reduce number of piers and consequently the delays and hazards inherent in maritime construction. After laboratory tests of locally available quartzite deposits, this material was approved for concrete aggregate. Experience on the job, where erratic results were frequently obtained, proved it advisable to add a certain proportion of dune sand, which was fixed at 50 per cent on the basis of tests. This sand acts as a lubricant and corrects the grading, increasing the proportion of fines. This method proved extremely practical and did not increase the costs. Field tests were made on 8-in. cubes. No especial care was given to the making of the specimens, so that the results may be considered as minimum values. Concrete in the arches was subjected to mechanical vibration in large masses, and is naturally far superior to that of the specimens tested. The following strength values were obtained on test cubes of the five grades of concrete used: Grade 1 (used in approaches), 2600 lb. per sq. in. at 7 d.; 3620 lb. per sq. in. at 28 d. and 5400 lb. per sq. in. at 1 yr. Grade 2 (used in approaches, apron, arches), 3500 lb. per sq. in. at 7 d., 4480 lb. per sq. in. at 28d. and 6300 lb. per sq. in. at 1 yr. Grade 3 (arches), 3860 lb. per sq. in. at 7 d., 5000 lb. per sq. in. at 28 d. and 6650 lb. per sq. in. at 1 yr. Grade 4 (foundations in sea water using fused cement, i. e., ciment fondu), 3180 lb. per sq. in. at 1 d., 4310 lb. per sq. in. at 7 d. and 4940 lb. per sq. in. at 1 yr. Grade 5 (foundations in sea water, ciment fondu), 4030 lb. per sq. in. at 1 d., 4810 lb. per sq. in. at 7 d. and 5190 lb. per sq. in. at 1 yr. The proportions by volume varied in the five grades as follows: 0.9 to 1.04 cu. yd. gravel, 0.5 to 0.65 cu. yd. of sand and 660 to 880 lb. of cement. The "Abrams cone" or slump test was used, the slump being 5.9 in. for portland cement specimens and slightly greater for fused cement. This test, however, loses its importance when the concrete is worked by vibration, as resultant settling is independent of quantity of mixing water. The portland cement used in 1:3 mortar had a compressive strength at 7 d. of 3080 lb. per sq. in. and at 28 d. of 3860 lb. per sq. in. The 1:3 mortar of the fused cement had a compressive strength at 2 d. of 3900 lb. per sq. in.; at 7 d. of 4320 lb. per sq. in. and at 28 d. of 4550 lb. per sq. in. A total of 32,500 cu. yd. of concrete was placed on this job, requiring a total of over 1600 tons of steel reinforcement and about 2600 cu. yd. of forms.—M. A. CORBIN

New Wandau bridge over river Enns (Austria). JOSEF KREBITZ. *Zement* (Germany), Oct. 9, 1930, V. 19, No. 41, p. 964-7.—Old wooden bridge was replaced by single span concrete arch bridge crossing highway, river and railroad. Main span over river valley is 242 ft. long, center being 50.8 ft. above normal water level. Arch is 16.4 ft. wide at center and increases to 23 ft. wide at abutments; its thickness varies from 2.6 ft. at center to 4.6 ft. at abutments. Some steel reinforcement was necessary at center and abutments only, since arch is so designed that only compressive stresses occur. Two prismatic concrete columns, 23.6 ft. wide and 6.5 ft. thick, are at ends of arch. Openings for highway and railroad are 32.8 and 45.3 ft. wide and 30.5 ft. high. Abutments consist of mass concrete foundations on solid rock. Roadway has inclination of 1 per cent; it is 16.4 ft. wide at center of arch and 19.7 ft. and 20.7 ft. wide over side openings. Reinforced concrete flat slabs, 16.5 in. thick with top layer of bituminous material, were used for its construction. Concreting was carried out in 31 sections, divided by joints 11.8 in. wide. Aggregates of high quality were found in neighborhood of construction site. All invisible concrete surfaces were finished with a coating of Inertol to prevent penetration of water.—A. E. BEITLICH

Reinforced concrete arch bridge over the river Rench in Erlach, Baden (Germany). H. GLOCKNER. *Zement* (Germany), Aug., 1930, V. 19,

No. 33, p. 782-4.—Old highway bridge, which was insufficient for increasing traffic was replaced by concrete arch bridge with span of 67 ft. Abutments of old bridge were used for new structure. Roadway, which is of suspension type, is 17 ft. wide and consists of 7.8 in. thick continuous reinforced concrete slab which is supported by 13 horizontal beams. These are 11.5 in. wide and 19.7 in. high and are placed 4 ft. 9½ in. apart. Cross-section of arches is 32.3 in. by 21.6 in. Two horizontal beams which are 16 in. wide and 19.7 in. high connect centers of both arches. Bridge crosses river at angle of approximately 76 degrees, 61 ft. above high water level. High early strength portland cement was used for arch construction while road slab was made of ordinary portland cement. Special care was taken for exact proportioning of aggregates which were sieve analyzed at site. Mixture (1:5) consisted of 1.5 parts gravel of 1⅜ in. size, 1 part gravel of ⅝ in. size, 2.5 parts sand and 1 part cement. Strength properties of cement and concrete were carefully examined during entire construction period.—A. E. BEITLICH

BUILDINGS

Transformer station in Nuernberg (Germany). G. PFEIFFER. *Zement* (Germany), Sept. 4, 1930, V. 19, No. 36, p. 855-8.—Building is five-story skeleton structure of reinforced concrete made exclusively of high early strength cement. It contains four transformer units. Foundation consists of one continuous reversed Pilz-slab, which is from 19.7 to 23.6 in. thick and strongly reinforced. Floors of superstructure were constructed for loads of 573, 471 and 164 lb. per sq. ft. Provisions were made for two additional stories on top when enlargements are necessary. These are designed for 123 lb. per sq. ft. each. Interesting construction details are given.—A. E. BEITLICH

Modern park structures in Frankfurt a. M. (Germany). CRAEMER. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 920-2.—Building which serves as an open air restroom was constructed of reinforced concrete. Flat ceiling was made as Pilz-construction with thickened center portion. Projecting part of ceiling rests on four reinforced concrete columns. Concreting was done in planed forms; no finishing was necessary. A second structure of similar construction is described.—A. E. BEITLICH

New forming methods for small concrete buildings. F. SCHNEIDER-ARNOLDI. *Zement* (Germany), Sept. 25, 1930, V. 19, No. 39, p. 922-4.—A building material with almost ideal properties for small building construction is light concrete with pumice or tufeceous materials as aggregates. Only disadvantage is price of forms which is too high compared with relatively small volume of concrete. Improvements were made by using certain standard forms for building parts, which were built very often. A new type of forms is described. Wire nettings are mounted on iron frames and are used instead of wooden forms. In order to keep exact distance between two sides of these wire forms, connecting links are placed between them. Concrete is poured with plastic or earth-moist consistency. Great care must be taken to keep concrete wet during setting period, since open wire meshes allow faster evaporation of mixing water, especially in warm weather. Amount of concrete which falls through meshes is very small. Distribution of concrete in forms can be easily controlled. Openings of the meshes are about 0.6 to 0.8 in. and thickness of wire is between 0.039 and 0.060 in. No finishing of concrete surface is necessary. —A. E. BEITLICH

Concrete distributed by two towers connected by adjustable chute. *Eng. News Record*, Oct. 23, 1930, V. 105, No. 17, p. 661.—Use of a chute suspended from a high line between two towers at opposite ends of a long theatre building was the economic solution of a concrete distribution problem

encountered in Beverly Hills, Calif. A small corner lot which dictated the location of the concrete plant at the inside rear corner and a high tower section at front of building necessitated this unusual arrangement. The building is 230 x 130 ft. in plan and 78 ft. high except for front section, which is 120 ft. high. Near the concreting plant a hoisting tower 247 ft. high was erected. From this point it was necessary to distribute concrete to the front corners of the building 250 ft. distant. A 187-ft. secondary tower without hoisting equipment was erected near the front of the building. The two towers were connected by a 1-in. cable which supported a 12-in. metal chute on a 1:3 slope. Secondary chutes extending from the towers enabled concrete to be distributed to all parts of the structure.—D. E. LARSON

Corrosion of structural steel. (See PROPERTIES OF CONCRETE.)

DAMS

Mersey river hydro development. CLIVE W. CURRIE. *Canadian Eng.*, Sept. 30, 1930, V. 59, No. 14, p. 501.—Nova Scotia Power Commission has completed and put in operation three developments on the Mersey River, at Upper Lake Falls, Lower Lake Falls and Big Falls, with a combined output of 28,000 h. p. at minimum head. Upper Lake Falls development consists of an earth fill dam with concrete core wall, 49 ft. high and 1600 ft. long with concrete spillway 250 ft. long. At Lower Lake Falls one mile below an earth dam with concrete core wall 4000 ft. long was built. Concrete spillway section is 850 ft. long. Dam at Big Falls is also of earth with concrete core wall with concrete spillway section 600 ft. long.—G. M. WILLIAMS

Unique cutoff construction and arched foundation features of Rodriguez dam. *Eng. News Record.* Oct. 16, 1930, V. 105, No. 16, p. 600-604.—Building a cutoff wall from streambed downward as excavation was advanced through shafts and providing an arch to support four buttresses of the highest Ambursen-type dam ever built, were innovations in foundation construction used at Rodriguez dam, now under construction on the Tijuana river in Mexico. The arch span at intradosal springing line varies from 86 ft. at cutoff to 76 ft. at downstream end of structure. This decrease of span has advantage of providing wedge action which increases resistance to sliding and places heaviest buttress loads over shortest span. Arch rise varies from 28.2 ft. at cutoff to 32 ft. at other end of 225-ft. length of arch barrel. Arch barrel is designed to carry entire load of four buttresses to sound rock at sides. For symmetrical loading, crown of arch was placed halfway between two buttresses. Final arch computations including rib shortening and shear distortion, proved entire arch to be in compression and no bending reinforcement was used. Heavy reinforcing parallel to and directly under each buttress provides for any lateral bending in the arch. Maximum design stress for concrete was 700 lb. per sq. in.—D. E. LARSON

The Perak river hydro-electric power scheme. *Engineer* (England), Aug. 22, 1930, V. 150, No. 3893, p. 191.—Perak River Hydro-Electric Power Co., has built a hollow reinforced concrete dam of the Ambursen type on the Perak River in the Federated Malay States, forming a storage reservoir having an area of about 10 square miles with a catchment area of 2350 sq. miles. Dam was built slightly curved upstream to take advantage of good rock foundation. Spillway section furnishing a head of 63 ft. at low water, has a total length of 684 ft. and is divided into two sections by a sector gate with free opening of 100 ft. situated between piers near the center of the river. Spillway section consists of reinforced concrete slabs forming a continuous deck on upstream and downstream faces, supported by reinforced concrete buttresses 18 ft. on centers based with horizontal concrete struts in every second bay. Slabs are held in place by offsets and notches in buttress heads. Vertical contraction joints are

provided at each buttress and horizontal joints are arranged at intervals of 16 ft. between each section of the slabs. On the left bank the dam changes to the solid gravity type with same spillway curve as hollow section. This section was poured with vertical contraction joints extending down to rock foundation at 30 ft. intervals. River bed below dam is protected from scour by reinforced concrete aprons extending 100 ft. downstream.—G. M. WILLIAMS

MISCELLANEOUS

Wagon for distributing ready mixed concrete. *Engineer* (England), May 23, 1930, V. 149, No. 3880, p. 584.—The "Sentinel" Waggon Works, Ltd., is constructing a wagon for distributing ready mixed concrete. The mixer drum with a capacity of 5 cu. yd., is cylindrical in section with a conical discharge end, and is filled with internal blades. Drum is mounted on a double-geared, tipping model steam wagon, with tipping gear operated by an oil pump. The drum revolves at 10 to 20 r. p. m. either when the vehicle is in motion or at rest. Two tanks mounted above drum carry water, one for supplying water for the mix and the other for flushing the drum after discharge. In operation, weighed cement and aggregates are loaded into drum through the apex of the conical end, proper amount of mixing water is sprayed in, and mixer drum is rotated about 5 minutes before arrival at destination.—G. M. WILLIAMS

New experiences in American and German concrete construction practice. LUZ DAVID. *Zement* (Germany), Aug. 7, 1930, V. 19, No. 32, p. 750-5.—Author discusses opinions and experiences of American investigators on manufacture of concrete, which were presented at annual meeting of American Concrete Institute in 1930. Special discussion is given of methods for manufacture of ready mixed concrete. Advantages and disadvantages of central plant mixed concrete and central plant proportioned truck mixed concrete are characterized. Cost calculations are given for economic installation and maintenance of ready mixed concrete plants. Second part of article deals with experiences in winter jobs. Heating and protecting methods for raw materials and finished concrete are illustrated. Calculation and design of reinforcement must be made subject of more careful consideration.—A. E. BEITLICH

Cooling tower at Croydon. *Concrete Constr. Eng.* (England), Oct., 1930, V. 25, No. 10, p. 555-57.—The most interesting feature from the point of view of construction is the arranging of the scaffolding for these hyperbolic cooling towers. The ultimate cost of the work largely depends on the scaffolding.—JOSEPH MARIN

Bonding new concrete to old. *Engineer* (England), Oct. 10, 1930, V. 150, No. 3900, p. 393.—The Department of Scientific and Industrial Research has conducted experiments to determine what precautions must be taken to insure a good bond between old and new concrete with the following results: (1) Laitance which forms a porous, chalky mass of non-coherent material must be removed. (2) If concrete is more than four hours but less than three days old the surface should be wire brushed and washed after removing laitance. (3) A ½-in. layer of mortar, without coarse aggregate, of the same composition as the new concrete should be applied before the new is placed. (4) If the concrete is more than 4 days old the surface should be chipped away, brushed and rinsed. This is followed by a slurry of neat cement, then a layer of mortar as in (3) before placing new concrete.—G. M. WILLIAMS

Full circle section of 17-foot tunnel lined in single pour. *Eng. News Record*, V. 105, No. 16, p. 610-612.—Cushman No. 2 tunnel, 13,000 ft. long and 17 ft. inside diameter, is located on the Olympic peninsula about 70 miles northwest of Tacoma, Washington, and will convey water from the Skokomish River through a spur of Olympic Mountains to develop a static head of 475 ft.

on hydraulic turbines of a 75,000-kw. plant at tidewater. A special tunnel form with pneumatic placing makes it possible to concrete full circle of lining 17 ft. in diameter and 40 ft. long in continuous operation. With this method of placing, copper water stops are required only in circumferential joints at 40-ft. intervals. The carriage for steel forms consists of two 5-ft. steel girders, 83 ft. long, supported on four end posts consisting of 8-in., H-section steel columns. Each girder carries along its upper flange a row of flanged wheels serving as rollers on which the circular steel form is advanced along the carriage when being moved ahead to new position. The cylindrical steel form is so constructed and hinged that it is readily collapsed for moving ahead and then expanded into position for placing concrete. All concrete is put into place by means of a 2-cu. yd. pneumatic concrete gun with a 6-in. delivery pipe about 80 ft. long. Thickness of concrete lining is specified as 15-in. average with 8-in. minimum.—D. E. LARSON

Selling concrete right from the mixer. *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 30-33.—The J. L. Shiely Co., producer of sand and gravel and crushed stone, St. Paul, Minn., has developed two plants for production of ready-mixed concrete. One plant adjoins sand and gravel plant and the other adjoins crushed stone plant, each one using material in which it is in close touch. A definite guarantee is made by the company to deliver a mix of specified strength; as for instance 1,500 lb. per sq. in. in 28 days for a 1:3:5 mix; 2,000 lb. for a 1:2:4 mix; and 2,500 lb. for a 1:2:3½ mix. The delivered prices for 1:2:4 concrete, which is mix used on most jobs (guaranteed to have a strength of 2,000 lb. at 28 days) is from $8.00 to $9.05 per cu. yd. on less than 100 yd. orders, depending on zone or distance from the plant. The 1:2:3½ concrete (2,500 lb. per sq. in. strength) is 15¢ per yd. more, and 1:3:5 concrete (1,500 lb.) is 75¢ per cu. yd. less. Orders of more than 100 cu. yd. carry prices 50¢ per yd. lower than those given on the price list. Heating in winter is charged for at rate of 40¢ per cu. yd. A discount of 5 per cent is allowed for cash or payment by tenth of the following month.—E. S. HANSON

Railroad box car with haydite concrete floor. *Rock Products*, June 7, 1930, V. 33, No. 7, p. 74.—Universal Atlas Cement Co., Buffington, Ind., has tried out a steel box car with a floor of haydite concrete. It has been found to have advantages over wooden floor for shipment of bulk cement, as power shovel or scraper works better on cement than on wood. The car was in a smashup which sheared the king pin of one truck but the floor was not injured. Floor concrete mix was 1 sack cement, 2 cu. ft. coarse haydite and 2 cu. ft. fine haydite. It was put on 4 in. thick but not anchored to steel floor, expansion joints and asphaltic fillers being placed between floor and sides. Round ⅜-in. reinforcing bars, spaced 6 in., running in both directions, were placed ¾ in. from bottom and wire netting was placed near top. Weight, about 36 lb. per sq. ft., or about two-thirds weight of ordinary concrete, and the cost was 35¢ per sq. ft. Advantages of the floor are greater strength than wood and better insulating qualities (when used in refrigerating cars). Disadvantages are greater weight and less nailability than wood. This is noticed with automobiles and like freight where cleats have to be nailed to bottom of cars.—EDMUND SHAW

Calibrating supports for exact centering of reinforcement. *Genie civil* (France), Sept. 27, 1930, V. 97, No. 13, p. 309-310.—The method recently invented by M. L. Ponsolle consists of small supports or clamps in the shape of a stirrup, star, disc, helix, etc., which are used on vertical and horizontal reinforcement alike and serve to keep the right distances between reinforcement and forms, as well as between individual parts of reinforcement. The method is successfully applied on a building under construction at Angers. Supports are made of very resistant metal which will withstand effect of working concrete, will not rust, and are furnished at moderate cost.—M. A. CORBIN

Concrete pipe testing machine. *Engineer* (England), Sept. 19, 1930, V. 150, No. 3897, p. 323.—A pipe testing machine has been made by Wm. Grice and Sons, Ltd., Birmingham. It consists of a platform built of steel sections mounted on four wheels. A hydraulic cylinder and ram, carrying a crosshead, is mounted on two steel columns. The ram and cross head are adjustable on the steel columns for any diameter pipe up to maximum specified. Load is applied through a hydraulic ram and pressure registered by means of a gauge.—G. M. WILLIAMS

Ferry on Lake Constanz between Constanz and Meersburg. THEODOR LUTZ. *Bautechnik* (Germany), May 9-23, and June 6, 1930, V. 8., No. 20-22-24, p. 259-9, 326-30, 350-3.—Difficulties were encountered during construction of harbors in Constanz and Meersburg. Quay is built of reinforced concrete. Three rows of concrete poles 12.6 in. square and 24.6 ft. long were driven. Poles of outer row are connected by dovetailed reinforced concrete piling. Piles are 23.2 in. wide, 6.3 in. thick and 20.7 ft. long. Distance between poles is 22.3 ft., and 4.9 ft. where expansion joints were placed. On this pile foundation rests reinforced concrete slab, 7.2 ft. wide, 2.5 ft. thick and about 328 ft. long with concrete railing on one side. Concrete piles were made at site of construction. Detailed description is given of arrangement of reinforcements, concrete mixtures, concreting and finishing of concrete.—A. E. BEITLICH

Attaining low cost in mixing and placing. CHAS. C. MOODY. *Concrete*, Nov., 1930, V. 37, No. 5, p. 23-24.—Office building for Inland Co., at Indiana Harbor, Ind., a 2000 cu. yd. job, with very low cost limits, was laid out and equipment provided. Concrete was divided into two parts: 500 cu. yds. in foundations and foundation walls, and 1,500 in the floor slabs. Sand and stone had two prices, one based on team delivery; the other on delivery by cars alongside of the building. Latter price was 35¢ per cu. yd. less. Delivery track was about 150 ft. from center of building. Crane with boom having drift of 40 ft., by which aggregate could be unloaded and dumped into hopper was available. Seventy feet remained between the hopper and building. Raising aggregate hopper from ground slightly higher than usual made it possible to place mixer closer to hopper and chute the mix directly into the hoist bucket, eliminating wheeling entirely. Hopper was put on extension legs sufficiently high to permit dropping the aggregates directly from measuring bins into mixer. Total labor cost was $104 each eight hours, and the average unit cost was close to 70¢ per cu. yd. Previously estimated cost of the concrete was about $1.50 per cu. yd., giving a total saving of about $1,600, from which were deducted cost of equipment handling. Total cost of concrete was thus only $1.19 per cu. yd.—C. BACHMANN

Finds pre-mixed concrete field made to order for gravel producers. *Pit Quarry*, Sept. 24, 1930, V. 20, No. 13, p. 58-60.—Keefner Concrete Co., Des Moines, Ia., has new ready-mix plant which utilizes aggregates procured, from the Des Moines River, by Hawkeye Co-operative Sand & Gravel Assn. which occupies an adjoining property. A drag scraper brings material from river bed into a sump whence it is pumped to the screen in the tipple. After washing and screening, the products are stock-piled. Beneath the stock-piles is a tunnel belt conveyor which delivers to an elevator which, in turn, delivers aggregates to respective sections of main hopper. Gravel batcher has a large dial scale with a dash-pot feature. Mixer is a 2-cu. yd. tilting unit. Concrete is delivered in bathtub bodies.—A. J. HOSKIN

Sioux City ready-mixed concrete plant was built for winter service. *Pit Quarry*, Sept. 24, 1930, V. 20, No. 13, p. 41-43.—Plant of L. G. Everist Co., Sioux City, Ia., is prepared to heat concrete in the winter season, loading it into delivery trucks at average temperature of 75° F. Live steam at a pressure of 50 to 60 lb. is fed into aggregates hopper above mixer through pipe with many holes. Water for concrete also is heated by a steam coil to a temperature of 180° F. When it does not pay to fire a steam boiler, oil-jet burner directs its

flame into mixer through a hole in charging chute. Cement bags are hand-trucked from cars, lifted by belt conveyor to batching platform, and emptied into batching hopper as needed. Bins for sand, gravel, and crushed stone discharge by gravity into batching hopper through lever-operated gates. Weighing is done with a 2-beam scale. Water is measured by a 100-gal. volumetric syphon tank. Concrete is dumped from 2-cu. yd. mixer directly into delivery trucks having bathtub bodies.—A. J. HOSKIN

Concrete units replace damaged brick-work in bridge pylons. *Concrete*, Oct., 1930, V. 37, No. 4, p. 17.—Walls of pylons on railroad viaduct, built 8 years ago with common clay brick in shape of hollow square and faced with Benedict stone, showed outward bulge due to expansion of brick backing, and replacement was necessary. First step was careful removal of cast stone top and facing. Material was lifted off with derrick, set on sidewalk, and carefully cleaned. In the meantime, brickwork was torn down and base of pylon cleaned. The 8 by 8 by 16-in. hollow concrete back-up units were set in 1:3 portland cement mortar, tempered with hydrated lime. Laboratory report on Van Houten units showed average absorption of 8.43 per cent; compressive strength of 1,280 lb. per sq. in. over the gross area and 2,040 lb. per sq. in. on the net area. Field test of clay unit showed absorption of 18.14 per cent.—C. BACHMANN

Nashville ready-mixed concrete plant stores combined aggregates. *Pit Quarry*, Sept. 10, 1930, V. 20, No. 12, p. 43-44.—Nashville Ready-Mixed Concrete and Supply Co., Nashville, Tenn., has plant with daily capacity of 400 cu. yd. New improvements will enable storage of six grades of aggregates in bins above mixers. Aggregates are handled by cranes to a hopper which feeds two bucket elevators, discharging into any of six compartments in a 250-ton steel bin. Bulk cement is unloaded to a ground hopper feeding a bucket elevator to a 2-compartment steel storage bin. A screw conveyor feeds from this bin to two 1-cu. yd. mixers. For one of the mixers, gravel is weighed in a 1-ton-capacity batcher, sand is measured in a 28-ft. inundator, and cement is weighed in a 1,000-lb. batcher. Materials drop into mixer hopper ready for next batch. Water is supplied from a tank alongside inundator. For other mixer, sand and gravel are handled by a 2-ton-capacity weighing batcher, cement by a 1,000-lb. weighing batcher, and water by a measuring tank. This second mixer prepares also lime mortar and cement mortar. Concrete and mortar are delivered in eight 3-ton trucks with agitator bodies.—A. J. HOSKIN

Delivering 60,000 cubic yards of centrally mixed concrete in month. R. P. BROWN. *Concrete*, Oct., 1930, V. 37, No. 4, p. 19.—Warner Co. of Philadelphia has established Central-Mix Concrete (CMC) as high standard by careful attention to all factors, including dependable delivery. Agitator type was decided best method of transporting and 15 four cu. yd. Clinton trucks were purchased at start. Fleet now comprises 85. Feature of equipment is reduction of entrained air. Unit consists of cylindrical tank horizontally mounted on truck chassis by trunnion bearings at both ends. Cylinder is kept constantly revolving so long as it carries concrete, by means of separate motor, there being no blades or paddles. "Certified concrete" is also furnished. Certification is made by independent testing and inspection company having absolute control of mix, checking of proportions and setting water tank. Inspector is stationed at each mixer, and verifies every batch. Inspection force checks sand and gravel at each mixer four times daily. At start of every job where "certified concrete" is furnished, inspection staff makes a set of not less than 6 standard cylinders and breaks them at 7 and 28 days. At Berks Street plant, sand and gravel are unloaded from barges to stockpile over tunnel feeding a 36-in. rubber belt conveyor which takes material to storage bins over mixers, which in turn feed measuring hoppers. Cement is purchased in bulk. One and one-half minutes mixing time is specified. Two Ransome 2-yd. mixers and one 3-yd. Smith tilting mixer are in constant operation being cap-

able of turning out from 1,500 to 1,800 cu. yds. daily. Orders are received over direct wire from main office after having been checked by engineering department. They are transmitted to concrete plant over a direct wire and to mixer control room over ticker tape, order stating mix, amount of concrete required, location of job, when delivery is to begin, and time between loads. As driver leaves mixer, his truck is sealed and drum started rotating. Product is sold either on strength basis or in accord with individual specifications. Deliveries are made on zone basis.—C. BACHMANN

Operates four pre-mixed concrete plants serving builders in St. Louis. W. E. TRAUFFER. *Pit Quarry*, Aug. 13, 1930, V. 20, No. 10, p. 39-45.—The Park Ave. plant of General Materials Co., St. Louis, Mo., is largest, with capacity of 1,500 cu. yd. daily. Its equipment includes locomotive cranes and 1-cu. yd. clamshell buckets, steel bins, screens, weighing batchers, with automatic dial scales, or weightographs, and three 2-cu. yd. mixers. Cement is added to the charge by hand from sacks. Fyler Ave. plant is the newest and all operations are mechanical with capacity of 900 cu. yd. daily. Sand and gravel are received in hopper-bottom cars and discharged into a long wooden bin with capacity of 350 carloads. Crushed stone is dumped by trucks into a concrete hopper whence it is conveyed into a section of main bin. Aggregates are delivered by conveyors to top of mixing plant and are fed, through a split chute, to two 4-compartment octagonal bins above mixers. Cement, unloaded in bulk from cars, by a power scraper, is elevated to bins with a capacity of 2,400 cu. ft. each. A screw conveyor feeds from these bins to a bucket elevator, to mixer department. Cement, sand and crushed stone are fed to two sets of weighing hoppers with dial scales. These hoppers discharge to batch hoppers of two 2-cu. yd. mixers. Clara Ave. plant resembles Fyler Ave. plant. Its capacity is 450 cu. yd. Aggregates are discharged from cars on a trestle to ground storage. Conveyors take the materials to a 4-compartment bin above the mixer. From its 4-compartment bin, cement is taken by screw conveyors to join aggregates which feed by gravity into weighing batchers which have dial scales and feed a 2-cu. yd. mixer. Branch St. plant is smallest with a daily capacity of 200 cu. yd. It has no mixers but prepares batches for mixing in transit. Aggregates are received in cars and unloaded by a crane to a 2-compartment bin with weighing hoppers. These feed to the truck mixers. Cement is received in bags which are opened into a small elevator discharging into a bin above the weighing equipment. A fifth plant is to be erected on Franklin Ave. at Twelfth St. The company operates 103 trucks.—A. J. HOSKIN

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Oct., 1930, V. 37, No. 4, p. 31.—Assuming a stairway of the type illustrated in the layout, the problem of form work may be considered as comprising construction of two reinforced concrete stair landings at levels intermediate between floors, with three short runs of stairs connecting the landings to each other and previously built floors. Stair landings are likely to be either a slab or of light-concrete beam—and slab construction supported on reinforced concrete posts; or several edges of the landing slabs may be supported on recesses in stair enclosure wall. Faces of top and bottom risers (at the connection with the previously built floors) are several inches in from the face of the hatchway opening in the floor construction. Form work for stair landings must be built first. The soffit panel is sloping panel that forms underside of the stair run. It is the first to be placed after form work for stair landings is in position. This panel, as a unit, consists of $\frac{7}{8}$ in. lagging placed parallel to the risers, and is held by $1\frac{1}{4}$ by 4-in. cleats. Longitudinal joists are supported on 2 by 6-in. ledgers spiked to 4 by 4-in. posts. Following assembly of the soffit panel and its supports, form work consisting of stringers, beam sides and risers must be placed. On the free edge of the stair run there may be a small beam extending from landing to landing or from landing to floor. This will require a form for the sides and bottom of this beam. In the more usual case there is no

beam extending below the soffit, in which event the soffit panel extends to and beyond the edge of the stairs and only a beam side is provided. On the wall side the stair concrete is to be in contact with the wall, so that there is no room for a stringer or a beam side plank at the side of the stairway. A 2-by-8 stringer is placed against the wall, just above the risers, scabs again being nailed against the stringer to hold risers. In designing a mix for a stair run, a good plan is to use a 1:3 cement-sand mortar, or a concrete mixture with small coarse aggregates (not exceeding the ½ in. size) for that part of the slab below the steel reinforcement. The rest of the slab and the steps themselves may then be made of ordinary concrete of dry consistency. Another practice is to use small-sized coarse aggregate, either pea gravel or crushed aggregate not exceeding the ½ in. size, for all concrete in stairway work. Most convenient place to set posts and props supporting stairway form work is on newly concreted stairway and landings immediately below.—C. BACHMANN

Concrete plastics in Afghanistan. (See ARCHITECTURAL DESIGN.)

Construction of quay at Rotterdam. (See ENGINEERING DESIGN—MISCELLANEOUS.)

Exposed surfaces of concrete and their protection. (See PROPERTIES OF CONCRETE.)

RAILWAYS

Tokyo's underground railway. H. SAKAKIBARA. *Eng. News Record*, Oct. 23, 1930, V. 105, No. 17, p. 655.—An underground railway 8½ miles long is being constructed at Tokyo, Japan, for the Tokyo Underground Railway Co. The subway consists of a double or twin tube having rectangular steel frames of I-beams spaced 5 ft. apart and embedded in heavy walls, floor and roof of reinforced concrete. Concrete in the walls and roof was poured through chutes from a portable mixing plant on the street surface.—D. E. LARSON

ROADS AND PAVEMENTS

Concrete roads and their joints. H. C. JOHNSON. *Engineer* (England), Sept. 26, 1930, V. 150, No. 3898, p. 349.—A concrete road built in Cork, Ireland, 10 years ago and subjected to medium heavy traffic shows no defects. Section is 21 ft. wide, 7 in. thick at center and 5 in. at edges, flat base, with the edges reinforced with a concrete curb poured later but doweled to the slab, and reinforced with 2 ½-in. round bars. Transverse steel reinforcement in slab consisting of ½-in. round bars, 15-in. centers, with 7½-in. longitudinal distributing bars, is placed 2½ in. above lower surface of slab at center and rises to 2 in. from upper surface at sides. Concrete was a 20 per cent mixture, that is, 20 per cent of volume of finished concrete was cement. Slab was laid in sections 80 ft. to 90 ft. long, one half day's run with transverse shoulder or supporting lip formed by digging a shallow trench at the end of the slab. Vertical joint between slabs consists of tar paper attached to a ¼-in. thick pine strip. Concrete slab was roughly leveled with shovels, smoothed with long screed with double handles, operated transversely, followed by a light roller longitudinally, then finally by a 10-in. belt transversely.—G. M. WILLIAMS

Expansion joints in concrete. *Canadian Eng.*, Oct. 14, 1930, V. 59, No. 16, p. 544.—The Technical Advisory Committee of the Ministry of Transport of Great Britain has made a survey of defects in a recently constructed road three miles long, 30 ft. wide between curbs, built on a ballast bank varying in depth from 2 ft. to 20 ft. Wearing surface consists of 2-in. stone filled asphalt carpet on 8-in. thick reinforced concrete foundation. Concrete was laid in bays 14 ft. 6 in. wide, full width of road with no center joint, on the alternate bay system, every fourth joint filled with bituminous sheeting, with joint

supported on a 12-in. by 8-in. beam formed below. Mix was 1:2:4 by volume, ballast aggregate, with 4.7 lb. reinforcement per sq. yd. placed 2 in. from under side of concrete slab. Traffic consists of 10,000 tons per day mainly fast moving heavy commercial vehicles. After twelve months of service cracks have appeared at 14 ft. 6 in. intervals, indicating movement between adjacent slabs. Concrete has failed at the joints apparently due to depression of slab carrying load below level of adjacent slabs which in turn are struck heavy hammer blows as the load comes on. Remedy for these joint movements on newly made ground has not been definitely established but defects may be eliminated by thickening slabs at joints, or as a whole, or by the use of beams or interlocked joints.—G. M. WILLIAMS

Granili Street in Naples. LUIGI BRUSCHETTI. *L'Industria Italiana Del Cemento* (Italy), Feb., 1930, p. 20-24.—Granili St. is a concrete paved street built in 1925 which carries a very heavy traffic of iron wheel horse drawn wagons. Although some repairs have been necessary recently, concrete has generally shown little signs of wear. The author contends that the fault was in the materials or method of placing at particular points where disintegration has occurred. It is suggested that the surface of a concrete street should be made up of about 2-in. stone and cement without sand or intermediate size stone. A photograph is given showing a bridge roadway of concrete which has been in place for 22 years without deterioration.—C. G. CLAIR AND M. N. CLAIR

Results obtained by use of cement in Germany. VILBIG, MAIER, MULLER, OTZEN AND RIEPERT. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, Washington, D. C., 1930.—First concrete roads in Germany were built in 1890. Although methods of building these "cement macadam" roads differed appreciably from those in use today, many of these pavements are still in good condition. Extensive use of concrete as a paving material is limited by economic conditions. In general, this type is used only on heavily loaded roads, roads carrying a preponderance of motor vehicles and roads constructed in new locations. Two course method is preferred, using a 1:7 mix by volume in base and a 1:4 mix by volume in top. Carefully selected, durable materials are used in top and local materials in base. For bottom course, concrete has a consistency of moist earth whereas concrete in top course is somewhat wetter to facilitate finishing. Both hand and machine methods are employed but machine finishing is on the increase. Pneumatic tampers are used to consolidate base course. Steel reinforcing has recently been introduced using a 2½-in. mesh, placed about 1½ in. below surface. Slab thickness varies from 6¼ to 12 in., of which about three fourths is in the bottom course. Thickened edge sections are employed, with longitudinal joints on all roads more than 20 ft. wide. Transverse joints are spaced from 26 to 49 ft. apart, depending upon conditions and are the plain butt or open joint type, subsequently filled with asphalt. Sodium silicate has been used for hardening surface, without any marked improvement being noted. Bituminous treatments for curing have not been successful, although original laboratory results were favorable. Curing at present is by means of canvas followed by earth or straw as in the U. S. Curing period is in general from three to four weeks. Rate of progress attained in construction is variable. From 395 to 430 ft. of pavement 16½ ft. wide in 8 hours is considered good performance. Results secured using special Soliditit cement (cement to which ground silica has been added), have been good. Results may have been due to care exercised during construction. Limited results from use of Rhoubenite concrete have not been satisfactory. This concrete contains "Houben powder" an admixture. At first, concrete construction met with considerable opposition in Germany, as it was felt that there was a danger of destruction of the slab by formation of cracks and by steel tires of horse-drawn vehicles. These objections are no

longer of importance because of increase in automotive traffic. Considering the results obtained with concrete roads, the outlook for this type of construction is steadily and surely becoming better.—F. H. JACKSON

Results obtained by use of cement in Belgium. E. VAN HAUWERMEIFEN. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Cement has been used in Belgium in two forms, in concrete and mixed with stone dust to form a macadam binder. Most of the pavements have been built under the "Rhoubenite" or "Soliditit" patents, being constructed between 1921 and 1927. Maintenance was at first neglected but is being attended to much more promptly than in the past. Maintenance consists in filling cracks and joints with tar or bitumen. Light bituminous skin coats have not proved successful. Due to practice in Belgium of constructing the even numbered slabs after odd numbered slabs have thoroughly set (sometimes a period of several days elapses) it is difficult to obtain smooth riding surfaces. Author believes slabs 33 ft. long should be constructed rather than 16½ ft. which is present practice, so as to reduce number of impacts due to difference in elevation between adjacent slabs. In contrast with American practice, concrete pavements are constructed on a plane subgrade and are thickened at center. Proportions are controlled very accurately, coarse aggregate being measured in 3 sizes. Proportions now required are as follows: 1000 lb. portland cement, 12½ cu. ft. of sand, 9 cu. ft. stone chips, .08 to .2 in. in size; 10½ cu. ft. stone chips, .2 to .8 in. in size; and 21 cu. ft. crushed stone, .8 in. to 1.6 in. in size. A relatively dry mix is used and the concrete is consolidated either by rolling followed by hand tamping or by mechanical tampers. Construction is in two courses. When rolling is specified, slabs are constructed continuously, otherwise alternate construction is employed. Curing is done by covering with sandy soil kept moist for 25 days. Specifications require that the concrete develop a crushing strength of 5650 lb. per sq. in. at 56 days. Tests are made on 4-in. cubes sawed from 16 in. by 16 in. blocks which are chiseled out of the pavement. A penalty is exacted for failure to obtain this value which is calculated as follows: Penalty in francs per sq. cm. $\frac{400-X}{100}$ where X is the unit crushing strength obtained by test (expressed in kg. per sq. cm.). In order to compare the results obtained with practice in the U. S. 6 in. by 12 in. cylinders are cast alongside the road. Concrete cores are also drilled from the pavement to compare with cubes and cylinders. Approximately 124 miles of pavement has been constructed on the national roads. This type has won the unanimous approval of the road user due to its cleanliness, non-skid properties, visibility at night, etc. The one disadvantage is irregularity of surface due to difference of elevation of adjacent slabs. A satisfactory estimate of cost is difficult to obtain. It may be said in general that cost of construction is about one-half cost of block paving and about twice as much as ordinary surface treated macadam. The report includes a discussion of dry cement-stone dust mixtures as macadam binders as well as so-called mortar macadams. In latter type, a rich portland cement mortar is spread upon a course of well consolidated crushed stone upon which a second course of stone is placed and the whole thoroughly consolidated by rolling until the mortar flushes to the surface. Author concludes that mortar macadam, although low in first cost, does not form a durable pavement, it being usually necessary to surface treat within one or two years. Construction by this method has therefore been largely abandoned in Belgium.—F. H. JACKSON

Results obtained by use of cement in Denmark. S. ELLERT, E. SUENSON AND P. ORUM. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Report describes a number of experiments in use of cement in road construction since 1924, including construction of a concrete pavement in accordance with methods used in U. S. as well as several sections of so-

called mortar macadam. Results of the trial section of concrete have been excellent. The cost of this type is so high, however, as to prevent its adoption in place of Durax (stone block) with which it has to compete. Various trial sections with cement macadam have been built. The methods include the use of both a dry cement-sand mixture and a cement grout with which to fill the voids in broken stone course. Use of dry mortars was not satisfactory due to difficulty in filling the voids completely. The results with grout seem to be reasonably satisfactory and the cost is considerably less than ordinary concrete. Warning is sounded as to comparative load supporting capacity of cement-macadam road and concrete road. With increase in traffic by motor truck, the time is probably not far distant when intensity of such traffic will require construction of concrete pavements and it is intimated that immediate economy of using more temporary road surfaces will hardly be economical in the long run.—F. H. JACKSON

Results obtained by use of cement in Spain. JOSE RODRIGUEZ SPITERI AND FRANCISCO DE ALBACETE. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Results which have been secured on a number of short sections of concrete pavement constructed in various parts of Spain are reported. A typical pavement on highway from Madrid to France through La Junquera has the following characteristics: Width, 23 ft.; thickness, 9 in. Proportions: cement, 760 lb.; sand, 16½ cu. ft.; gravel, 29 cu. ft.; water, 4.2 to 5 gal. per 7 cu. ft. of concrete. Expansion joints were 48 to 64 ft. apart, and the pavement was cured by ponding, except a small stretch cured with Hunt process. This pavement was constructed in 1928 and is giving satisfactory service. More cracks are reported in section cured by the Hunt process than in the water cured section. Report also describes so-called mosaic concrete pavements as constructed in Spain. These roads consist of a concrete base, 6 in. thick, a cement mortar course 2½ in. thick and a top course of small stones with portland cement mortar binder. The diameter of the stones in the surface course varies from 3 to 5 in. on upper side and height is from 5 to 3½ in. The bottom area is 75 per cent of the surface area. These pavements are in reality stone block wearing surfaces embedded in a cement mortar cushion and laid on a concrete base. They appear to be giving excellent service under traffic carrying a substantial portion of steel tired horse drawn carts.—F. H. JACKSON

Results obtained by use of cement in France. R. BOUTTEVILLE. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Development of cement concrete road has been retarded in France by reason of high cost. At end of 1928, approximately 605,000 sq. yds. of the highway system had been surfaced, out of a total area of 1,030,000 sq. yds., including city streets. Indications are, however, that the amount of paving of this type will rapidly increase within the near future. As the result of test and experiment since the war, the methods of construction have become fairly well standardized. The report covers the various details of design and constructon as now practiced in considerable detail of which the following is a brief summary: Concrete pavements are either laid upon old stone road, reshaped or upon a foundation of stone or ordinary concrete. Thickness of slab varies, depending upon conditions, from 4 to 8 in., with a tendency to use the thicker sections. Pavements are in general of uniform depth, although an increased edge thickness of from 2 to 4 in. is sometimes used. Longitudinal joints are only used in pavements over 25 ft wide. Transverse joints are placed from 32 to 130 ft. apart with a tendency to the shorter lengths. Although recognizing the advantages of laying slabs in alternate sections from the standpoint of crack width, the practice has not been followed in France due to construction difficulties. The use of reinforcement is exceptional; only being used to increase resistance to bending on very unstable ground. Tests having shown that resistance to wear

can be greatly increased by increasing the amount of cement and also by careful selection of aggregate the old practice of one course construction has been abandoned and a two course type adopted with a proportion of approximately 550 lb. of cement to 17½ cu. ft. of sand and 35 cu. ft. of stone for the lower course as compared to a mix having 1200 lb. of cement (usually super-portland) to 10½ cu. ft. of sand passing a ¼-in. sieve and 35 cu. ft. of hard crushed porphyry, diorite, etc. Considerable attention is paid to the grading of the aggregates. Concrete is mixed in central mixing plants and transported to the road in small carts or wheelbarrows. About 40 gal. of water to 35 cu. ft. of concrete is used. The concrete is compacted by pneumatic or vibrating hammers or plates several types of which are described. The consolidated concrete is finished by means of a longitudinal roller weighing 1100 lb., about 18 ft. long and 10 in. in diameter. The methods of compaction described have resulted in much denser and stronger concrete with a perfect bond between the courses. The report estimates the life of concrete roads as now constructed to from 10 to 15 years. Attention is called to the fact that some of older roads are now in need of surfacing and describes a pneumatic device for scarifying worn concrete surfaces. The author does not believe that concrete roads should be built unless they can be constructed to resist surface wear. Need of more detailed specifications for concrete pavement work is discussed.—F. H. JACKSON

Results obtained by use of cement in England. J. B. L. MEEK, D. A. DONALD, S. H. MORGAN AND R. A. B. SMITH. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Report discusses the theory of designing concrete pavements as well as the materials used and construction methods followed in England at present time. Effects of temperature, moisture and traffic upon design are discussed and necessity of artificial drainage in order to lower moisture content of subsoil is stressed. The theory of loads is discussed to some extent, attention being called to the fact that loads as high as 30 tons per axle have to be provided for. Widening pavement 18 in. or 2 ft. beyond actual wheel path is practiced rather than thickened edge as used in the United States. Reinforcing is used on unstable soils, design varying with type and character of traffic. Slabs are usually reinforced both top and bottom. A minimum thickness of 8 in. is indicated for all main roads. Construction of slabs in longitudinal strips about 10 ft. wide is recommended with corresponding transverse joints 20 to 30 ft. apart in mass concrete and from 50 to 60 ft. apart in reinforced concrete. The alternate bay method is sometimes used, especially where it is desired to pave entire width at one operation and tampers and finishing boards cannot be used transversely. Due to necessity of providing a wearing surface of high resistance, pavements are usually constructed in two courses, using local materials in the base and crushed granite in the wearing course. The difficulty of applying a bituminous carpet which will adhere is discussed. The desirability of providing a carpet coat to take wear is noted however, and attention is called to the fact that application of a prime coat of tar or creosote will assist the carpet to adhere. Construction methods follow in general those used on the Continent. A dry mix is employed which is consolidated by tamping and cured by means of a 2-in. layer of sand kept wet. High early strength cements as well as, in certain cases alumina cements are employed when it is desired to open the pavement to traffic at an early age. The present day cost of concrete pavement in England varies from 8s to 10s per sq. yd., unreinforced but including excavating, etc. A method of constructing concrete roads known as the "sandwich" system is described. This consists in laying a course of broken stone, covering this with a plastic 1 to 2 mortar over which is spread another course of broken stone and the whole rolled with an 8-ton roller until the mortar flushes to the surface. This method has progressed well beyond the experimental stage and may be considered satisfactory. The cost varies from 5s to 7s per sq. yd. for a 4½ in. depth.—F. H. JACKSON

Results obtained by use of cement in Italy. L. DELL GUADIO, F. COLAMONICO AND I. VANDONE. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Use of concrete as a paving material in Italy dates back about 18 years. Concrete roads of the Soliditit type now comprise about 218,000 sq. yds. A notable development has been the use of concrete on the roads which are built to carry automobile traffic exclusively. A total of 1,570,000 sq. yds. have so far been constructed with about 1,210,000 sq. yds. to be laid in the near future. Concrete in "motor roads" is proportioned, mixed and placed in accordance with American practice. No steel tired traffic is, however allowed on these roads. For the main roads it has been found necessary to adhere to the European practice of two course construction, using a dry mix which can be tamped into place and hard wear resisting aggregates. A major portion of the report is devoted to a description of an experimental road constructed in 1929 which was built for purpose, primarily of developing means for constructing one half of the total width at a time so as to leave other half open for traffic. Other variables studied included the construction of alternate sections; proportioning by weight instead of by volume; using different grades of coarse aggregate; using 5 different cements; using various proportions of cement from 660 to 990 lb. per cu. yd., using various consistencies, using various consolidating and finishing methods, using single course as compared to two course work and using various thicknesses of slab. The above form 108 different combinations, each section being 15 ft. long and 11½ ft. wide. In order to accelerate the results of this test, an artificial steel tired traffic of 180,000 tons was carried over it. This did not produce enough wear to permit of a satisfactory comparison, which is now being obtained by means of very accurate level readings on plugs set in the pavement. These tests are being supplemented by profilometer tests to determine surface irregularities, in much the same way as is done in the United States. The results of this experiment are being awaited with great interest by the authorities.—F. H. JACKSON

Results obtained by use of cement in Holland. P. W. SCHARROO AND J. W. CLERY. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—This report, after calling attention to the inherent advantages of concrete as a paving material, calls attention to the fact that good results can only be secured when great care is used in construction. Only specialists and contractors who can provide proper personnel and good materials should be employed. Open bidding for this reason is out of the question. Concrete paving in Holland is still in beginning of its development. Unstable soils present great difficulties to this as well as other types of construction. During years 1925 to 1929 about 47 miles of concrete had been laid on the national system of roads as well as a small amount in the cities. The theoretical discussion of factors which affect density of concrete, importance of avoiding oversanded mixes is stressed as is also the necessity of using hard, strong and clean well graded aggregates. Use of dry mixes which have to be tamped is advocated in order to increase density. Ordinary portland cement will give as good results as super-cement. Although practically all roads in Holland are built in two courses, authors favor one course construction, on account of difficulty of properly bonding two courses together. Resistance to wear is of great importance and increases with strength and density. Water content should be kept low. Bituminous carpet coats have not proved satisfactory. Water glass is used as a surface treatment to increase hardness. Authors doubt the value of the treatment in case of concrete of proper density. Practically no reinforcement was used until 1928. Experiments, however, with longitudinal bars have indicated value of reinforcing of this type. Most recent practice as to joint construction has been to follow American practice, with tongue and groove longitudinal joint and transverse joints supported at ends with dowels fixed in one slab, but free to move in the other. Authors however,

are not entirely satisfied with this method, believing that joints should be so constructed as to leave slabs entirely free to move with respect to each other. They doubt that this is the case in American practice. Conclusion is drawn that concrete roads if properly built are better adapted to intense traffic of heavy vehicles than any other type.—F. H. JACKSON

Results obtained by use of cement in Sweden. E. PAUL WRETLIND. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Modern cement concrete roads now extend all over south portion of Sweden. During period 1925 to 1929 a total of approximately 15 miles was constructed, all single course construction. In 1926, a special specification covering a high grade portland cement was introduced. This cement known as Class A is used in public road construction. The requirements for strength at 7 days are almost double that of ordinary portland cement. Report describes an experiment with the alternate bay type of construction, in which openings 1.6 ft. wide were left at end of each 100-ft. slab and these openings filled with reinforced concrete about three weeks after the construction of the long slabs. In order to preserve alignment the reinforcing was doweled into the ends of the long slabs, with proper provision for slippage. Curing is done with wet sand, earth and straw. Sodium Silicate is used as a surface hardener without, however, any appreciable benefits.—F. H. JACKSON

Results obtained by use of cement in Switzerland. E. WYDLER. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—Very few concrete roads were built in Switzerland before 1929, due, first, to high cost and, second, to fact that political sub-divisions of country into 25 Cantons is not favorable to establishment of a unified system of roads. Approximately 105,000 sq. yd. of concrete was laid in 7 Cantons in 1929. General practice is to lay concrete on old well compacted stone bases. First pavements were constructed in two courses, using a rich mix and very hard aggregate in the top course to minimize surface wear. Results are not satisfactory due to excessive cracking of the top course. Present practice is to use either one course or two courses, using same proportion in each. In accordance with continental practice, a very dry mixture is used which is either tamped in place with pneumatic tampers or finished with a finishing machine. It is concluded that longitudinal joints should not be used when width is less than 20 ft. Transverse joints should be constructed about 33 ft. apart. Trouble has been experienced with joints due to abrasion and impact. It is concluded that joints should be filled absolutely flush with the road surface with an unyielding material.—F. H. JACKSON

Results obtained by use of cement in the United States. FRANK T. SHEETS, ROY W. CRUM, E. M. FLEMING, A. N. JOHNSON, CLIFFORD OLDER AND R. E. TOMS. *Proc. 6th Congress*, Perm. Internat. Assoc. of Road Congresses, 1930.—At end of 1928 there had been constructed in the United States on strictly rural highways approximately 59,000 miles of portland cement concrete pavement, or more than four times mileage which existed at end of 1920. During year 1928 alone, 8,756 miles were built. Such intensive construction has necessarily called for the rapid development of design and construction practices. This report outlines the present methods employed in design, construction, and maintenance of concrete pavements. Considerable advance has been made in structural design of pavements. Report calls attention to certain fundamentals which should be observed in construction of subgrade, and reference is made to new developments in design of cross section, as well as important features in proportioning of reinforcing steel and construction of expansion joints. The trend is towards intensive research as to effect of various characteristics of constituent materials on quality of concrete. Recent investigations indicate that characteristics of the aggregates have more influence on tensile and flexural strength of concrete than upon compressive strength.

The design of concrete mixtures by the water-cement ratio and mortar-void methods is rapidly gaining in favor. To secure benefits of proper design of mixtures, the ingredients must be accurately measured and water content closely controlled. These important features are described and references given to more complete information. Rapid strides have been made in development of equipment for handling materials and for mixing and placing concrete. Rigid inspection and modern equipment have aided in securing improved quality, smoother surfaces, and better production. The curing problem still needs much study. The rapidly increasing mileage of concrete pavement has stressed the problem of maintenance. Much study has been given organization, methods and equipment. Some of more important methods are described. One new development is the use of high early strength concrete for repair work.—F. H. JACKSON

Final conclusions reached by Permanent International Association of Road Congresses. *Proc. 6th Congress*, Washington, D. C., 1930.—(1) Cement is becoming generally used as a paving material and has many inherent advantages. Rapid hardening cements have special advantages in particular circumstances. (2) Cement has been used successfully in construction of cement concrete base courses for other surfaces, for cement concrete pavements, and for cement-bound macadam. (3) Cement concrete pavements and also cement concrete base courses protected by appropriate wearing surfaces are suited to heavy traffic. (4) Where a large volume of steel-tired traffic is encountered, if cement concrete is adopted as a pavement material, a two-course pavement with the upper layer composed of very hard aggregates should be used instead of a single-course pavement. (5) Single-course pavements have successfully carried maximum volumes of traffic and maximum wheel loads when the traffic was largely rubber tired. (6) Cement-bound macadam has been successful on roads carrying light traffic not inimical to the macadam type of construction. This method seems especially advantageous in locations where the conditions of drainage or exposure are unfavorable to the use of ordinary waterbound macadam. A protective wearing surface seems equally indispensable on cement-bound and water-bound macadam. (7) In designing cement concrete pavements and cement concrete base courses to be surfaced with other materials the resulting pavement should have equal load-carrying capacity or structural strength, when similar traffic conditions are to be met. (8) Competent engineering supervision of design, construction, and maintenance of cement concrete pavements is necessary to insure good results. (9) It is desirable that subgrades be uniform and stable. (10) Pavement slabs must be designed to carry expected loads. Edge thickening is advantageous as a means of producing an economic and balanced structural design for concrete pavement slabs. (11) Longitudinal and transverse joints are commonly used and must be designed to meet traffic, subgrade, climatic conditions, and shrinkage of concrete, but in view of the fact that a number of concrete roads have been successfully constructed without joints, it is advisable that further research should be made on the whole subject of joints and cracks. (12) Scientific design of concrete mixes and weight proportioning of aggregates represent the most modern practice. (13) Construction operations are performed mostly by machinery, with resulting lower cost and better workmanship. Value of a cement concrete roadway depends to a large extent on perfect execution of work and, in particular, on homogeneity of the concrete. (14) Thorough curing of concrete surfaces is essential. (15) Maintenance of concrete surfaces, properly constructed, is relatively simple and reasonable in cost. In particular, maintenance should comprise immediate filling with suitable material of expansion joints and of any cracks which may occur.—F. H. JACKSON

The experimental street of Binasco. ITALO VANDONE. *L'Industria Italiana Del Cemento* (Italy), Feb. 1930, p. 8-15.—An experimental section of

concrete road was constructed along Pavese St. in Binasco in connection with the study of the problem of developing a type of paving that could be placed over the existing substructure and which would economically resist the action of both automobiles and horse drawn traffic. About 180 sections 11.5 ft. wide and 14.8 ft. long covering 108 different conditions were poured. The variables included 5 different crushed stones, one gravel, 5 different cements, 4 proportions of cement, 2 to 6 different consistencies, 2 depths of paving and two types of paving (single and double layer). Concrete was placed in half road width sections, the other half of road remaining open to traffic. Concrete was mixed at a central point and conveyed to place by a narrow gauge railway. Very careful attention was given to control of various quantities entering into the mix. The sand and gravel were weighed. Gravel was separated into sizes to promote uniformity. Difficulty was encountered in controlling the water content with the usual water tank on the mixer. Samples of concrete were taken at time of pouring for laboratory test but it was considered that complex disintergrating action of the steel wheel animal drawn traffic could be only properly shown by a large scale test under real traffic. Numerous horse drawn wagons were therefore circulated over the experimental section for several days. The wear was concentrated in a narrow lane so that a condition equivalent to one year of normal traffic was simulated. Very little wear resulted except for the removal of the surface skin. Measurements of profile of pavement were taken by a special apparatus which makes a continuous record as the machine moves along. It is planned to repeat measurement at intervals to learn something about the way in which deterioration of pavement occurs. Economics of use of concrete roads in Italy is discussed and it is concluded that this type of road can be properly used when it costs no more than other types when all factors of cost including deterioration and maintenance are considered.—C. G. CLAIR AND M. N. CLAIR

WATER WORKS

Water tower in Hassloch i. Palatinate (Germany). *Zement* (Germany), Oct. 16, 1930, V. 19, No. 42, p. 999.—Foundation of structure consists of one circular reinforced concrete slab, 34.4 ft. in diameter, heavily reinforced radially and circumferentially. Cylindrical stair case (4.9 ft. in diameter), in which pipe lines for feed and discharge are located, is erected in center of foundation slab. Tank consists of two concentric containers, each of which has capacity of 132,000 gallons. Outer diameter of tank is 52.5 ft., its height is 24.6 ft. and depth of water is 23 ft. It is carried on a circle of 12 reinforced concrete columns connected by two horizontal concrete rings each placed at one third of total height of columns. Tank is faced outside with hollow cast concrete bricks to protect structure against temperature effects. Ceiling is 4.7 in. concrete slab. Total height of structure is 144.4 ft.; 390 tons cement, 1265 tons sand and gravel and 126 tons steel were used for its construction.—A. E. BEITLICH

SHOP MANUFACTURE

Achieving light-weight building unit with ordinary aggregate. D. R. COLLINS. *Concrete*, Nov., 1930, V. 37, No. 5, p. 20-22.—The standard size 12-in. product of block manufacturer at Cedar Grove, Wis., weights 7 lbs. less than the 8 by 10 by 16 in. unit he manufactured a year ago. Corresponding saving in weight has been made in the 10-in. unit. Saving is due to substitution of rectangular for oval cores—taking material out of the corners and thinning the webs. The manufacturer is making units 7¾ in. high by 15¾ in. long. His average 8 in., 10 in. and 12 in. units weigh from 42 to 44 lb., from 51 to 53 lb., and from 60 to 62 lb. respectively. When a thin-webbed unit is being made it

requires careful supervision of the mix and the manufacturing operation, and should be handled only in a high-grade plant having a reputation for high-grade products.—C. BACHMANN

Proposed German standard specifications for light cast stones made from pumice. *Zement* (Germany), Sept. 11, 1930, V. 19, No. 37, p. 879-80.—Three different types are specified: stones made from pumice aggregates with (1) hydraulic or dolomitic lime, (2) cement and other hydraulic binders and (3) with only or mostly cement or special cements. Form of units must be 10 in. long, 4.7 to 6.3 in. wide and from 2.5 to 5.5 in. high. Compressive strength must be for units made with lime, 170 lb. per sq. in.; with cement, 284 lb. per sq. in. and with special binders 427 lb. per sq. in. Average of 10 tests must be determined. Weight per cu. ft. should not be higher than 52.95 lb. Heat conductivity number for air dried units should range from 0.13 to 0.16 at 68° F. Units must stand freezing and thawing 25 times. Additional data are furnished explaining origin and geologic nature of raw material, chemical composition, porosity, physical structure and possibilities for application in construction practice.—A. E. BEITLICH

What is a fair market price for light-weight units? H. H. POTTS, *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 16-17.—The material considered is a concrete unit for backing and partitions, and may be of any of light-weight aggregates now on the market. Their value is considered in relation to clay tile and complete data worked out shows what the cost of concrete units would be to compete with clay units at various market prices: for example if 5 by 8 by 12 in. clay tile are delivered to the building site at a price of $65.00 a 1,000, then 8 by 8 by 16 in. light-weight concrete units are worth $170.00 a 1,000 delivered. The author then works the problem still further to show that light-weight units of this size can be sold at a price of $170.00 per 1,000 or 17¢ each and carry a profit of 2¢ each.—E. S. HANSON

Two-in-one concrete pipe plant at Beloit, Wis. *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 12-15.—The plant of the Illinois-Wisconsin Concrete Pipe and Tile Co. at Beloit, Wis., occupies the site of a worked out sand and gravel plant and is separated into two departments, one for making pipe up to 24 in. in diameter of the bell and spigot type and the other for making pipe from 21 up to 60 in. of the bevel joint type. Plant also manufactures larger sizes by hand. Handling of products has been worked out very efficiently by means of two travelling cranes and several small tractors with trailers, also tractors with caterpillar tread for pulling and pushing larger sections.—E. S. HANSON

Random ashlar walls. W. D. M. ALLAN, *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 34-36.—This concluding article of a series of three on concrete random ashlar describes the manufacture of the various concrete masonry units required for typical random ashlar wall construction. Almost any plant equipped to manufacture standard concrete masonry units usually can, with only a few changes in existing equipment, manufacture units which can be used in building random ashlar walls. For typical wall construction, seven sizes of units, in which the dimensions of the exposed faces determine the sizes, will permit the flexibility ordinarily required in designing random ashlar patterns. Inasmuch as concrete random ashlar walls usually are built as 4-in. veneers or 8 in. masonry walls, this discussion deals with manufacture of units 4 and 8 in. thick. However, the operations are practically identical for producing units having other wall thicknesses.—E. S. HANSON

Materials

Admixtures

Densifying of mortar and concrete against dampness and water under hydrostatic pressure. Hugo Leube. *Betonwerk* (Germany), May 18, 1930, V. 18, No. 28.—The use of densifying admixtures discussed. Admixtures for this purpose are divided into three classes, namely: liquid, paste and powder. Each is analyzed as to proper mixing technique.—H. Frauenfelder

Aggregates

An accelerated soundness test. Dr. Herbert F. Kriege. *Civil Eng.*, Nov., 1930, V. 1, No. 2, p. 120.—Detailed consideration is given to sodium sulfate soundness test as applied to concrete aggregate (fine and coarse), filter bed sands, and coarser mineral aggregates commonly employed as sewage disposal media. Although this test dates back to 1828, it has not been fully or uniformly controlled. The essential elements in the test are outlined and explained and detailed procedure is suggested. Of thirteen cooperating laboratories, the eight laboratories that followed suggested procedure closely, correctly classified ten kinds of slag and stone into sound, unsound, and questionable groups. Much of suggested procedure represents a consensus of opinion from the answers to 99 questionnaires sent to prominent engineers in the fields of highway, sanitary and testing engineering.—H. J. Gilkey

Gasoline tractors and trailers in limestone mine. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 64.—Centropolis Crusher Co., 8 miles from Kansas City, Mo., mines limestone by room and pillar method. Rooms are 30 ft. wide with 25-ft. pillars. The level is such there is no upgrade to the crusher. Air hammer drills put in 16 ft. holes which are loaded with 40 per cent powder. Top cut is taken and remaining bench is lifted. The working height is 16 ft. Loading is by 1-yd. shovel with 12-ft. boom and 7-ft., 8½-in. dipper stick. It is equipped with an electric light outfit including a 250-watt floodlight. Haulage is by two tractors and side-dump trailers. Distance from face to crusher is about 500 ft. Two trailers deliver 1,200 tons per 10-hr. day and gasoline consumption is about 80 gal. Between shifts tractors and trailers haul fines to dump. Direct charges for labor, fuel and oil approximate 3¢ per ton. The crushing plant, of conventional design, produces these sizes: 2-in., 1¼-in., ¾-in., ½-in., buckshot and fines. The output is sold in considerable part to make ready mixed concrete.—Edmund Shaw

Influence of dust content in aggregates on weight per cu. ft. and compressive strength of concrete. G. Kathrein. *Oesterreichische Bauzeitung* (Austria), Oct. 25, 1930, V. 6, No. 43, p. 705-7.—Additions of limestone powder of different finenesses were made to aggregates, and their influence on mortar and concrete properties examined. High early strength cement was used for preparation of 2¾-in. cubes of 1:5 cement : sand mixture with additions of 4, 10 and 20 per cent limestone powder of 2 different finenesses. Compressive strength and weight per cu. ft. were determined after 7 days air storage. It was found, that additions of stone powder have favorable influence on mortar strength, which increases about 50 to 85 per cent with additions of 4 and 10 per cent. Addition of 20 per cent lowers strength. Fine powder has greater influence than coarse particles. Weight per cu. ft. shows similar behavior, increasing with 4 and 10 per cent and decreasing with 20 per cent additions of stone powder.—A. E. Beitlich

Standardization vs. discrimination. Stanley Hands. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 46-48.—The practice of proportioning concrete

by a definite grouping of a limited range of sizes of aggregates appeals to the engineer because it relieves him of much detail and he is not concerned with losses of production. Fact that other methods of proportioning would produce concrete of same quality is not important to him. If, however, he has to produce his own aggregate he will adopt methods of proportioning not based on accepted standards but on natural laws covering use of aggregates as he may find that his pet proportions create undue waste. Discrimination in this case will be favorable to more use of run-of-the-pit. Standards may change but fundamentals and natural laws do not change. The application of fundamentals may make it undesirable that some things should be standardized. Some concrete workers think that the art is working toward discrimination in use of materials. Most of fundamentals are ready. Only cooperation of producer is needed. Utility of aggregates is not to be considered from standpoint of *what* they will make alone but *how* they will make what is intended. Measure of their utility lies in profitableness of their use. The surface property of an aggregate may be an important factor. It is generally known that some surfaces become covered with a gelatinous film that is often transparent and not easily detected. Material which may qualify under grading and sizing standards may be found undesirable for lack of bonding quality. Research has not determined all conditions and proportions that make differences in materials. Until adequate facts are known it would be undesirable for the industry to set up standards. All aggregate associations have exceptional research organizations and it is very desirable that their work be continued.—EDMUND SHAW

Effect of flat and elongated particles in mineral aggregates. STANTON WALKER. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—The purpose of the report is to cite whatever of definite information is available on the effect of flat and elongated particles and to suggest procedure for investigations. Although specifications have commonly drawn discriminatory lines against such material, there are little data upon which to base definite requirements. Research conducted by the National Crushed Stone Association and the National Sand and Gravel Association indicate that flat and elongated particles in the amounts to be expected in practice (10 to 15 per cent) do not have deleterious effects upon workability, strength, finish and quality of surface of concrete. Some tests reported by Gilkey on specially prepared aggregates where the coarse aggregate consisted entirely of flat particles indicated some weakness. It is concluded that further investigations are required before generally acceptable conclusions can be drawn and suggestions for procedure are made.—HIGHWAY RESEARCH BOARD

Uniform description of graded materials and evaluation of their fineness. M. SPINDEL. *Tonind. Ztg.* (Germany), Nov. 2, 1930, V. 54, No. 88, p. 1385-6; No. 89, p. 1401-3.—Surface areas of aggregates are constant for any given fineness modulus so that any change therein gives a direct measure of work performed during grinding. A metric fineness modulus is proposed with the sieve openings $10°$, 10^1, 10^2 . . . 10^5 microns as basis with each range divided into five fractions with intermediate sieves the log. of whose openings shall be 1.0, 1.2, 1.4, 1.6 . . . 5.0, which then correspond to the fineness moduli of gradings of broken solids whose mean sizes correspond to the several sieve openings. This system has the advantage of being applicable to cement and other fine materials.—F. O. ANDEREGG

Bethlehem Mines' newest crushed stone plant. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 33.—At the Steelton plant of the Bethlehem Mines Corp., which produces concrete aggregate, flux and furnace lining dolomite, the sizes of stone to be sold as aggregate are thoroughly washed. Scrubber is 6 ft. dia. and 20 ft. long and capacity of washing plant exceeds 200 tons per hour. The rotary screen has a 6-in. pipe with 52 $\frac{5}{8}$-in. holes for washing the material

received from the scrubber. Additional sprays play on the outside of the screen. Only fresh water is used in the screen and after use there it is sent to the scrubber. The plant is all steel and concrete construction.—EDMUND SHAW

Crushed stone producer noted for originality builds sand plant. *Rock Products*, Oct. 11, 1930, V. 33, No. 21, p. 41-45.—Sand and gravel plant of Dolomite Products Co., Rochester, N. Y., owned and operated by John Odenbach employs an unusual method of handling with a much simplified washing and screening plant. The material is excavated by a 12-in. centrifugal pump on a steel dredge. It is boosted to help it through a 600-ft. pipe line by a 10-in. pump in a permanent concrete building. The full lift is 138 ft. The screening plant has a double-deck vibrating screen below a steel sluice box for the first separations, 2½-in. square meshes on the upper deck and $\frac{3}{16}$-in. square meshes below. Oversize goes to a No. 5 crusher with no return to the screen. The intermediate product goes to a 3-deck vibrating screen to be finished. It has 2¼-in., 1¼-in. and ½-in. meshes and all products go to pockets. The minus $\frac{3}{16}$ in. from the primary screen goes to automatic sand cones. The pockets for holding material are of concrete set below the ground level. They are discharged by a high speed travelling crane with 2½-yd. clamshell bucket, the runway being 40 ft. above rails. Bucket discharges into ten 65-ton steel bins for truck loading or into 3 compartment, 150-ton batching bins. Only steel and concrete are used in construction.—EDMUND SHAW

Light concrete for construction of 400 dwelling houses in Berlin-Zehlendorf. (See FIELD CONSTRUCTION—BUILDINGS.)

Reinforced concrete construction. Part I. Raw materials and their production. (See FIELD CONSTRUCTION.)

Relation between durability of concrete and durability of aggregates. (See PROPERTIES OF CONCRETE.)

CEMENT

Discussion on "A tentative specification for high alumina cements." *Structural Eng.* (England), Nov., 1930, V. 8, No. 11, p. 380-384.—The tentative specification which appeared in Oct. 1930, issue of Structural Engineer is discussed in considerable detail.—V. P. JENSEN

Portuguese standard specifications for cement. *Tonind. Ztg.* (Germany), Oct. 16, 1930, V. 54, No. 83, p. 1319.—Cements of "lower strength" must have at least 2850 and 243 lb. per sq. in. after 7 days water storage and 4300 and 300, compressive and tensile respectively, after 28 days in water. Cements intended for use in sea water must meet the above requirements when stored in sea water and must have at least 5000 and 330 lb. per sq. in. after 84 days similar storage.—F. O. ANDEREGG

Ashes absorbed by cement clinker. H. MOSCHITZ. *Tonind. Ztg.* (Germany), Nov. 13, 1930, V. 54, No. 91, p. 1430-1.—Dust removed from a cement kiln working with thickened slurry was about 3 per cent, most of which escaped from the stack. The clinker takes up about 40 per cent of ash.—F. O. ANDEREGG

Soluble salts in portland cement. *Betonwerk* (Germany), April 13, 1930, V. 18, No. 22.—The efflorescence sometimes noted on concrete and cast stone is discussed and its formation described. Cause is given as water soluble salts in cement. Method of determining these salts in cement is given and a suggestion for preventing efflorescence by use of barium oxide, which transforms soluble salts into insoluble.—H. FRAUENFELDER

German standard specifications for portland cement, iron portland cement and blast furnace cement. *Tonind. Ztg.* (Germany), Oct. 9,

1930, V. 54, No. 81, p. 1292-6.—These 3 cements are placed on a par, definitions being given for each. The composition of slag from which the two latter are ground with portland cement clinker is defined: $\frac{CaO + MgO + 1/3\,Al_2O_3}{SiO_2 + 2/3\,Al_2O_3} > 1$

Fineness is 2 per cent on No. 30 (75 mesh per in.) and 25 per cent of No. 70 (176 mesh per in.). The initial set shall be not less than one hour. The strengths 1:3 with German normal sand and with 8 per cent water must be at least:

Lb. Per Sq. In.

	Standard Cement		High Early Strength Cement	
Storage	Tensile	Compressive	Tensile	Compressive
3 days in water	...	....	357	3570
7 days in water	257	2570	...	....
28 days in water	...	3930	...	....
28 days combined	430	5000	570	7150

The tensile test is not regarded as of as much value as the other. Full directions are given for methods of testing.—F. O. ANDEREGG

Perchloride acid method of silica analysis—Its advantages over double evaporation. WALLACE K. GIBSON. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 70.—The perchloric acid method takes only 30 to 45 m., according to the water that has been added to the solution, saving much of time that has to be spent on double evaporation of silica and further baking of dry residues. Care must be taken not to boil with ordinary dilutions as an explosion will occur before the boiling temperature is reached. If it is desired to boil, the solution should be about 150 cc. with a 0.5 g. sample. The article gives the perchloric acid method in detail with figures showing the silica contents found by this and other methods.—EDMUND SHAW

Improved grinding methods at the Diamond Portland Cement Co. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 51.—At the Diamond Cement Co.'s plant at Middlebranch, Ohio, three 7 x 24-ft. compeb mills were formerly used in grinding raw material. One of these was put in closed circuit with a 14-ft. Sturtevant air separator and after a year's trial of this combination a second mill was similarly equipped. These two mills were then able to grind all the raw material needed. Without air separators each mill produced from 17 to 18 tons per hour. With air separators each produced 22 to 23 tons per hour. Material going to the separator from the compeb has 52 per cent minus 200 and 73 per cent minus 100. The finished product removed by the separator has 88 per cent to 89 per cent minus 200 and 97 per cent minus 100. Returns to the mill from the separator have 15 per cent to 16 per cent minus 200 and 46 per cent minus 100. Total circulating load is 46 tons per hour. Each separator takes 35 h. p. to drive. They are fed by steel encased elevators and the rejects are returned to the mill mixed with the primary feed.—EDMUND SHAW

Further developments in the application of dry fine grinding with air classification. HOWARD G. WRIGHT. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 57.—It has been ascertained that it is more important in preparing material for cement manufacture to eliminate coarse oversize than to produce much "flour." Air classification with grinding is shown to give excellent results in holding the particle sizes within narrow limits. Separation made by reversed current system of air classification used with Hardinge mills is in three stages. Air blown into the mill through a trunnion is reversed and drops the coarsest particles in the mill. The same air issuing from the mill passes through a rotary separator where the medium-coarse particles are removed. Then it passes to a "superfine" air classifier which takes out all but the product desired, this being caught in a cyclone. Oversize particles from rotary and from "superfine" classifiers falls into air tube through which air is blown into mill

and are thus returned to the mill. Records show that power consumption is not much if any greater with air separation than without, added output of the mill making up for the power used by the classifier. A lower ball consumption has been noted. A feed of uniform quality is necessary for best results and installation of a constant weight feeder in one case gave 2.5 tons per hour added mill product.—EDMUND SHAW

Tentative specification for high alumina cements. Prepared by the Institution of Structural Engineers. *Structural Eng.* (England), Oct., 1930, V. 8, No. 10, p. 369-373.—Specification includes method of manufacture and composition of the cement, samples for testing, tests of fineness, chemical composition, compressive strength, setting time, and soundness. Sections deal with non-compliance of tests, copies of vendor's tests and analyses, and the Appendix gives a tensile test for acceptance purposes only.—V. P. JENSEN

Weight of slurry. E. TAYLOR. *Rock Products*, Nov. 8, 1930, V. 33, No. 23, p. 64.—The author shows how he derives the following formula for the weight of one cubic foot of wet slurry: $W = \frac{6243\ s}{100 - a + sa}$ in which W is the weight in lb., s the specific gravity of the dry slurry and a the percentage of water in the slurry.—EDMUND SHAW

Free lime, constancy of volume and strength. Report of research laboratory of the Association of German Eisenportland Cement Companies, Duesseldorf (Germany). A. GUTTMANN. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1078.—Based on own experiments, author agrees in general with findings of G. Haegermann (c. f. *Zement*, Oct. 16, 1930, V. 19, No. 42, p. 982-4) concerning relation between volume constancy and free lime content of portland cement clinker. Presence of free lime has tendency to lower strength of clinker which contains much tricalcium silicate, but is able to improve strength of clinker, which is rich in dicalcium silicate, in later ages.—A. E. BEITLICH

Overcoming heat losses in cement manufacture. GEOFFREY MARTIN. *Rock Products*, July 5, 1930, V. 33, No. 14, p. 42.—The author leads up to a discussion of his own design of a flotation kiln by describing the heat losses in a rotary kiln and comparing with a shaft kiln which is more economical of fuel. He emphasizes that as clinker cannot be made at a lower temperature than 1370°C. (2498°F.), the kiln gases must have a higher temperature to impart this heat. But too high temperature cause high radiation losses. With coal firing a gas temperature of 2600°F. may be obtained. Higher temperatures if they were possible would be more efficient. Costly failure of British experiments at Swanscombe was due to ignorance of these facts. Hot exit gases were introduced into the kiln in an effort to save heat but gas temperature was cooled below 2500°F. and no clinker was formed. The great loss of high grade heat is internal radiation. Various methods to prevent this loss have been suggested and most promising of these is a cloud of spray or dust formed of raw materials in either a wet or a dry condition. These experiments are still going on. Author's flotation kiln, an improvement on design of Sir Percy Girouard, employs dust cloud principle mentioned. The kiln is a cavity worked out of a limestone cliff and it is of such a shape that the gases will circulate vertically. Raw material dust is fed into downwardly flowing side of this circulation and carried to bottom where there is a zone of intense heat. Dust falls through this and becomes clinker. It is discharged by conveyor in tunnel below. Gases after dust has fallen from them rise on other side of cavity and flow across to pick up more dust and descend.—EDMUND SHAW

Contribution to the question of fine cement grinding. ERNST RISSEL. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1079-80.—Two brands of cement were ground to different finenesses and effect of grinding was studied by measuring water permeability of 1:6 mortar test specimens and com-

pressive strength. One fact developed is that fine grinding affects water permeability to a far greater extent than it does compressive strength. Based on this evidence author points out that examination of water permeability must be considered of greater value than compressive strength tests for valuation of cement obtained by fine grinding. Further tests dealing with very fine cement fractions are under way.—A. E. BEITLICH

Influence of mixing water on mortar strength. E. PROBST AND K. E. DORSCH. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1009-11.—Standard compression and tensile strength tests were made with one high early strength cement, two ordinary portland cements, one blast furnace slag cement and one high alumina cement. Mixing water was in one case ordinary service water and in another case distilled water. Samples mixed with distilled water showed higher compressive strength and lower tensile strength than samples mixed with service water. Variations are different for different kinds of cement. Standardization of mixing water for standard test methods is suggested. Investigation of influence of storage water on strength properties showed no difference for different types of water.—A. E. BEITLICH

Lime and magnesia determination in portland cement raw mix. G. W. JORDAN. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 62-63.—Objections to usual acid-alkali method are: Iceland spar used as standard may contain as much as 1½ per cent impurities; color change of phenolphthalein covers a pH range from 8.4 to 10.2; $Al(OH)_3$ is soluble in excess of NaOH and Mg $(OH)_2$ is hydrolized during the continuation of the titration for magnesia alone. These errors neutralize one another but it does not follow that the net result is correct. The author recommends using solutions exactly 2/5 N HCl and 1/5 N NaOH, standardizing on pure (Bureau of Standards) potassium-acid phthalate, as an alkametric standard, with phenolphthalein as an indicator. One gram of raw mix is dissolved in a 200 cc. graduated flask with 50 cc. of 2/5 N HCl, the flask attached to condenser and boiled. While still hot enough to exclude CO_2 excess acid is titrated with the 1/5 N NaOH, methyl red being used as indicator. 0.05 cc. is added to NaOH reading for difference in pH value of methyl red and phenolphthalein as indicators. Corrected reading is subtracted from 100 to give lime equivalents of lime and magnesium carbonates. The solution is then brought to a boil and 50 cc. of N/2 neutral sodium oxylate solution is added to precipitate lime. The whole is diluted to 200 cc. and filtered. 100 cc. of filtrate, which should be alkaline to methyl red but neutral to phenolphthalein is titrated with a dilute alcohol-glycerine solution of potassium palmitate which is just alkaline to phenolphthalein. Red end point with phenolphthalein shows when all magnesia is precipitated, 0.3 or 0.4 cc. being subtracted. This is amount necessary to give a red color with a blank solution just alkaline to methyl red. One cc. N/10 potassium palmitate equals 0.002 grams MgO. No more time is needed than for the ordinary method and author believes results will be much more accurate.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part IV. GEOFFREY MARTIN. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 53-55.—The author shows how he derives the following formula for kiln output, which he believes to be original, $M = 1.263 \frac{S}{V} d^2$ tons per hour, M being output, V the volume of exit gases per lb. of clinker made and S their terminal speed and d the diameter of the kiln. For good practice, M equals $0.1123d^2$ tons per hour. For unusually good practice M equals $0.153\ d^2$ tons per hour. These are with wet process kilns. With dry process kilns M should equal $0.1356\ d^2$. While the author was with the British Association of Portland Cement Manufacturers the work of a number of kilns was compared with what it should be by this formula. Improvements in practice were thus introduced that gave large

savings, in one case amounting to $75,000 a year. It appears from this formula that kiln production may be increased either by reducing the volume, V, of gas per lb. of clinker or by increasing its terminal speed, S. Hence reducing fuel consumption per lb. of clinker or reducing exit temperature will give a larger output. The author has patented the use of fans for increasing the terminal speed S of exit gases. Where it is desirable that the speed should be lower in certain parts of the kiln baffles are placed in these parts.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part V. GEOFFREY MARTIN. *Rock Products*, Oct. 11, 1930, V. 33, No. 21, p. 58-59.—To make clinker raw materials must be heated to 1371 C. To attain this temperature kiln walls must be heated to a higher temperature but not high enough to fuse brick work lining. Assuming that all heat necessary to make clinker passes through clinkering zone, then surface area of clinkering zone must radiate away absolutely as much heat as passes through the clinkering zone per unit of time. From the above, the formula for surface of kiln in clinkering zone is derived: $S = 315.7 \frac{M}{y}$ sq. ft. where M is the output in long tons of clinker per hour and y is pounds of clinker produced per pound of fuel. In an example y is assumed to be 3, corresponding to a fuel consumption of 33 per cent and required surface area works out to 105.2 M sq. ft. A table appended gives the surface areas of clinkering zones for outputs of 1 to 10 tons per hour. To find required length, L, of the clinkering zone: It has been shown that diameter of kiln, d, is $2.98\sqrt{M}$, and since $L = S/\pi d$, L works out to $11.24\sqrt{M}$. From these the size of the clinkering zone for any output of a rotary kiln may be calculated. A table is appended giving dimensions for outputs from 1 to 45 tons per hr. and for diameters from 3 ft. to 20 ft.—EDMUND SHAW

Constitution of cements. NORBERTO P. COSTA. *Doctoral Thesis*, National University of Buenos Aires (Argentina), 1930, 32 p.—Author presents a detailed microscopical study of three classes of cements. Clinkers or unpulverized cement were submitted to different methods of cooling after having been heated in an oven to 2597° F. for portland cement clinkers and to 2732° F. for aluminous cements. The methods of cooling were in the oven, in air, in sand, in oil, in water and in ice. Plates for examination were made by placing pieces of clinker into hot Canadian balsam. After cooling, these plates were ground and polished. Argentina portland cement obtained by dry process, one high early strength German portland cement obtained by wet process, and one French cement, high early strength, were used. Work is based on a study by Doctor Hendrick. Some of objects of thesis were to study the very great influence of large crystals in the clinker, its size, its grouping around a nucleus and its effect on the strength of cements. These characteristics were attributed to two fundamental operations in making a good cement, namely, heating and cooling. Conclusions by author are briefly: That a clinkerized cement is not simple mixture of aluminates and silicates, but it is very probable that, in its constitution, they form partly solid solutions of isomorphic mixtures of silicates of calcium and of iron, with aluminates of calcium. That the fused cements are formed by a mixture of aluminates, silicates and silica-aluminates of calcium, mixed with slag of silicates of iron. Aluminates seem to crystalize in the body of the slag, while silicates and silica-aluminates crystalize in the vitreous mass (vidrio) of their own composition. Predominant element in these cements is $5CaO, 3Al_2O_3$ in its unstable form. Portland cement obtained by wet process has same proportion of constituents as portland cement obtained by dry process, but in the first case most of the crystals of Alit and Celit are uniform and of somewhat larger size. Grouping of grains of Celit around a nucleus of Alit were observed. This was not observed in cements from the dry process. (Celit—solid solution of di-calcium aluminate in di-calcium silicate. Alit—solid solution of tri-calcium aluminate and tri-calcium silicate.) In

portland cement of high initial strength—obtained by the dry process—it is observed that proportion of Celit is greater than in other cements and grains of Celit and Alit are larger. Therefore, high strengths of a cement are a consequence of crystalline groupings, and of size of crystals, above all those of Alit. Most interesting form is grouping of crystals around a nucleus of other crystals, in a radiated spheroidal form. In portland cements, quenching, or cooling, causes a variation in some physical characteristics of crystals, but its chemical constitution remains constant. However, in fused cements, action of cooling changes chemical constitution. Many micro photographs and a two-page bibliography are included.—N. H. ROY

Investigations on protection of concrete against aggressive solutions. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Austrian tests on concrete columns with high quality steel reinforcements. (See ENGINEERING DESIGN—MISCELLANEOUS.)

PROPERTIES OF CONCRETE

The analysis of the solid concrete body, a means for production of concrete mixtures of great value. ERICH WEISE. *Die Bautechnik* (Germany), May 2 and 16, 1930, V. 8, No. 19-21, p. 284-7, 315-8.—Cementing properties, voids, cement content, water-cement ratio and compressive strength of concrete mixtures were studied and subjected to mathematical calculations. It is pointed out, that excess of water must be avoided. Decisive for properties of finished concrete is water content, which fact is expressed by w/c-ratio. Amount of water which adheres to aggregates is of great importance. Above a certain limit, an excess of water will be of less influence, when used for cast concrete. A number of formulas for calculation of valuable concrete mixtures are derived.—A. E. BEITLICH

Research comes out of the attic. WYATT BRUMMITT. *Concrete*, Dec., 1930, V. 37, No. 6, p. 28-30.—Industry today has taken over duties of research with same motive as that of struggling scientist of last generation. Portland Cement Association laboratory at Chicago is typical of industrial research. Work was begun with no definite goal but recognition of need for larger knowledge of cement and concrete. One of first projects was establishment of suitable measuring stick to gauge phases of work. Strength assumed of vital importance and effort made to determine effect of cement content, quantity and proportion of aggregate, and mixing time. Water content considered secondary until experiments led to water-cement ratio law. Three classes of investigations now interest researchers: Continued analysis and refinement of cement-producing process; perfected technique of using concrete under "impossible" conditions; and widening range of uses. Department, headed by F. R. McMillan, occupies half of 5 story building, employs 50 men, and maintains fellowship for research at Washington. Findings of quarter-million experiments are on file.—C. BACHMANN

The question concerning the water impermeability of concrete. VOGELER. *Bautechnik* (Germany), May, 1930, V. 8, No. 23, p. 342-3.—Author discusses directions, which are given in book: Specifications for mortars and concrete (Anweisung fuer Moertel und Beton; cf. *Zement* (Germany), Jan., 1930, V. 19, No. 3, p. 64, *J. of the Amer. Concr. Inst.*, April, 1930, V. 1, No. 6, Abstr. Sect. p. 68) concerning relations between aggregate gradation, mixing water and water permeability of concrete. Ostendorf gives his experiences with the preparation of concrete of high water impermeability and outlines further investigations in this line.—A. E. BEITLICH

Collapse of a lock in Huentel (Germany), investigation and repairs. RUDOLF JEHN. *Bautechnik* (Germany), June 27 and July 18, 1930, V. 8, No. 28-31, p. 438-41, 478-81.—Investigation of concrete and local conditions revealed that sewer water and ground water contained only small amounts of harmful sulphates and magnesium salts but very great amounts of aggressive carbon dioxide, which fact, together with porosity of concrete led to its destruction. Bottom slab of concrete showed stratification and layers of sound and disintegrated concrete. Repairs of lock consisted of removal of old concrete bottom and replacement by new slab. Concrete with an addition of trass was used and was worked to a very dense structure. It was provided with heavy reinforcements.—A. E. BEITLICH

Analysis of complicated concrete mixes. J. L. HEINTZMAN. *Rock Products*, Nov. 8, 1930, V. 33, No. 23, p. 62-64.—Author says that his results obtained by methods given have been upheld in court in suits for infringement of Straub cinder block patent. After explanation of method for qualitative examination to determine coarse aggregate used and some other things a method is given for analyzing concrete of cement and cinders. The sample is digested with 50 cc. 10 per cent HCl and insoluble determined. As cinders are 95 per cent insoluble and cement is about 98 per cent soluble, the insoluble found divided by 95 will be practically the percentage of cinders. Where sand, cinders and cement make the concrete, insoluble found as before is dried and weighed. Then sand is separated by Thoulet's heavy solution in which cinders float while sand sinks. With concretes of cement sand and slag, some of slag is removed and analyzed for insoluble, soluble silica, and lime and apparent specific gravity and loss on ignition are determined. The proportion of slag may be figured from either soluble silica or lime in the concrete and also in the slag. A similar method is used with concrete of cement, sand and limestone, a sample of the limestone being removed and analyzed separately. The limestone content may be found by formula which includes the CO_2 contents of the concrete and the limestone. With concretes of unknown composition there must be a careful visual examination and the use of physical methods, which are described, in addition to chemical analysis.—EDMUND SHAW

Relation between durability of concrete and durability of aggregates. F. H. JACKSON. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Basic requirements for durability are: the use of sound and durable aggregates, and maximum resistance to the entrance of water. The aggregates play a double role since the range of sizes plays an important part in producing impermeability. The characteristics of aggregates which affect their durability in concrete, and the effects of other factors such as permeability, water absorption, etc., are not well understood, and thorough investigations are needed. Concerted laboratory studies looking toward the formulation of standard tests for durability, and field investigations to correlate laboratory and structural experience are urged. Outlines for research work are presented.—HIGHWAY RESEARCH BOARD

Special investigation—curing of concrete pavement slabs. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Progress report setting forth the facts established by study of the available data with respect to strength, surface scaling, and volume changes as affected by the various well known curing methods, together with the tentative conclusions drawn by the committee. The report will include complete summaries of the data collected by the committee.—HIGHWAY RESEARCH BOARD

Durability of concrete in highways and highway structures. C. H. SCHOLER. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—General discussion of the factors involved . . . Types of disintegration observed as evidence of a lack of durability . . . Report of laboratory

and field investigations of durability as affected by method of testing, size and shape of specimens, method of sampling, character of aggregate, quality of cement paste, permeability and absorption as a factor in disintegration, and effect of water-proof coatings. An accelerated freezing and thawing test is presented as a means of testing the resistance of concrete and concrete aggregates to weathering agencies.—HIGHWAY RESEARCH BOARD

Concrete dams in Norway. *Report*, Norwegian Society of Engineers, July, 1930, 272 p., Den Norske Ingenioersforening, Oslo (Norway).—To investigate effect of water action when under hydraulic pressure on dams of concrete, the Norwegian Society of Engineers in Oslo have made thorough examination of 88 of more important concrete dams in Norway, some of which are more than 30 years old. These investigations were started in the summer 1926, and this is first report by committee. It was found that very few of the dams were uninjured and that injuries were not evenly distributed over dams, thus showing that proportions of concrete mixture must have varied during their construction. Joints had most of the injuries. Organic matter, iron and manganese substances were accumulated on the injured parts. Though none of the dams was found to be very seriously damaged, committee advised repair of injuries. Main reason for injuries is said to be chemical action of water. Thus water will penetrate cracks caused by temperature changes and frost and start to remove soluble parts of concrete. Flood water is known to be more or less acid and is therefore lime dissolving. Concrete mixture was found in most cases to be too lean, and it is pointed out that sooner or later a concrete structure under hydraulic pressure would be completely destroyed by this acid water. Use of membranous water proofing is suggested for protection against temperature cracks but no means has been found to protect this layer against mechanical action of water. Interesting suggestion proposes placing thin watertight concrete slab in front of dam, to make concrete dam dry by keeping air space between slab and dam. This type construction was used for Ringedal dam, 100 ft. high. Slab was 8 in. thick at top and 18½ in. thick at bottom, supported by columns with spiral reinforcing. Air space between slab and dam was 6.6 ft. Horizontal spacing of columns was 7½ ft. and vertical spacing varied from 10 ft. to 6 ft. at bottom.—ODD ALBERT

Investigations on protection of concrete against aggressive solutions. OTTO GRAF. *Zement* (Germany), Oct. 2, 9, 16, 30 and Nov. 6, 1930, V. 19, No. 40-41-42-44-45, p. 936-41, 970-4, 995-8, 1041-3, 1066-8.—Best protection of concrete is obtained, when aggressive waters are kept away from concrete. This fact is very often not sufficiently taken into consideration although it may be possible by certain changes of design of structure. Special shaping of structure may also lead to same effect. Running water has in general greater effect on concrete than standing water. When concrete is only partly immersed in aggressive water, dividing line between water and air is subjected to greatest corrosive effect. Water is raised in concrete body by capillary attraction, and salt contents in upper portion is increased by evaporation. Studies were made in order to find cements most resistant against sulphate waters, waters with great contents of uncombined carbon dioxide, certain salt solutions and organic media. Iron portland cement with low lime content loses less strength than normal portland cement when immersed in linseed oil. After storage in 10 per cent solution of magnesium sulphate, iron portland cement lost in 15½ months 18 per cent of its 28-day strength, while high early strength portland cement and blast furnace slag cement showed gains from 1 to 9 per cent. In magnesium chloride solution, cubes of high early strength cement were destroyed, iron portland cement lost 33 per cent and blast furnace slag cement gained 6-7 per cent in strength. Degree of fineness is very important: finer cements show less disintegration than coarser ones. Various cements were tested in different mixtures with sand and different consistencies and stored under magnesium sulphate and magnesium chloride solutions. Further sub-

ject of investigation was effect of cement content and gauging water on resistance against movement of liquids in concrete body. Graduation was studied which gives greatest resistance against chemical attack. Mixtures of 1:3 cement:sand were more resistant than 1:6 mixtures. Amount of mixing water should not exceed absolutely necessary limit. Presence of 3 per cent clay in sand caused great losses in strength when specimens were placed in aggressive solutions after a seven day moist closet storage, but retrogression did not take place when specimens were dried for 35 days before submitting to test. Additions of trass seem to improve concrete resistance. Finally influence of storage and finishing on concrete durability were studied.—A. E. BEITLICH

Fire damage in Riga. ED WEISS. *Beton Eisen* (Germany), Oct. 5, 1930, V. 29, No. 19, p. 344-348.—Buildings 134, 135 and 218 of Prowodnik Rubber Company, leased to the Soviet Government for flax storage suffered severe fire damage on April 11, 1930. Units 134 and 135 are five-story reinforced concrete structures, each three bays wide and fifty-eight bays long, measuring 92 ft. x 874 ft. in plan. Unit 218, a one-story structure with steel roof trusses, and measuring 99 ft. x 874 ft. in plan occupies the 99 ft. space separating these two buildings. Structures were erected in 1913 through cooperation of German and Swiss construction company. There is no indication of faulty construction but there is much to criticise in design. Beams and columns were evidently figured as non-continuous units with no regard for frame action—usual practice in Russia at that time. These units were tied together at points with small amount of steel and with small haunches. Design would not have been permitted according to Russian standards of 1914. Long spans and light sections give impression of weakness. Temperature joints divide structures into lengths of approximately 130 ft. It is surprising that wind forces for which no provision was made have had no visible effect throughout the 16 years during which buildings have been exposed to severe sea-storms. There were no fire-walls in any of three structures. At time of fire, flax to a depth of 10 ft. with open aisles along the columns and exterior walls, was stored on lower 3 floors of Unit 134, in Unit 218 and on ground floor and part of second floor of Unit 135. Fire was combatted by 30 hoses and 10 fire boats. An effort was made to flood the flax but to avoid striking hot concrete with cold stream. Steel beams, with lower surfaces exposed for attachment of shafting, reached white heat and concrete was heated in some cases to red glow. Columns in burning stories displayed a progressive outward bowing until collapse of structure. A deafening rattle like machine gun fire, perhaps due to failure of reinforcement, immediately preceded collapse. A total length of 620 ft. of Unit 134 collapsed after estimated exposure to fire of from 4 to 6 hrs. Portions still standing seem to indicate that cement was burned out of the concrete but that sand and gravel suffered no damage. In many cases column steel bulged and concrete scaled off, often for considerable depth into the core. The horizontal cracking and outward bowing evident in exterior columns at floor levels may be due largely to fact that these columns were comparatively fixed against lateral movement, at the foundation and at upper stories (where no flax was stored). Heating of slabs to 1110° F. and to 1830° F. would lead to lateral expansion of 4.5 in. and 7.1 in. respectively, at intermediate floor levels. These lateral forces would be sufficient to crack columns and to force them outwards. Destructive effect of a stream of water on hot concrete is shown by a photograph in which path of stream is indicated by a deep gouge in slab soffit and entire removal of concrete for one foot length of beam. The 9500 short tons of flax lost are valued at $1,750,000 while building damage is $250,000. Some insurance experts have expressed opinion based on this fire, that steel covered with brick or tile is more fireproof than reinforced concrete. Structure itself contradicts this conclusion for lean concrete used in upturned portion of spandrels broke as a unit at limits of various pours but showed little surface damage. A few brick walls on ground floor were completely destroyed. Mortar was burned out and the bricks burst.—A. A. BRIELMAIER

Small movements in concrete. (See ENGINEERING DESIGN—MISCELLANEOUS.)

ENGINEERING DESIGN

BRIDGES

Pneumatic caisson foundations for Japanese bridge. J. MASAKE. *Eng. News Record*, Nov. 27, 1930, V. 105, No. 22, p. 848-850.—A handsome concrete-arch bridge with stone facing has been built to replace old wooden bridge across Shinano River at Niigata, a city on north side of main island of Japan. The bridge consists of six spans of hingeless concrete arches, two each of 154 ft., 151 ft., and 142 ft.; in addition, there are two 48-ft. two-hinged land spans. A headroom of 18½ at low water is sufficient for the craft on the river. The riverbed is composed of layers of fine and coarse sand to depth of 80 ft. Pneumatic caissons were used for piers and abutments of main bridge. Sand from caissons was used in making concrete, along with river gravel and Japanese portland cement.—D. E. LARSON

Vierendeel bridges. E. BALIS. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation in *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 632-34.—In Belgium the Vierendeel girder is widely used and shows an economy of about 20 per cent over ordinary steel lattice structures, though when reinforced concrete is used the saving is not so great. The actual stresses in these bridges are about 6 per cent less than theoretical stresses. A closer agreement would require laborious calculations. Two classes of this type of bridge are in general use: (1) the bowstring and (2) the arch with monolithic decking.—JOSEPH MARIN

Reinforced concrete bridge at Wisbech. *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 609-12.—This reinforced concrete bridge now in the course of construction over the river Neme, at Wisbech, will be the largest span portal type bridge in Great Britian. Clear span will be 92 ft. 6 in. and width between parapets will be 45 ft. Choice of suitable design was influenced by considerable rise and fall of tides at bridge site. The maximum rise in tide is 15 ft. The type of construction adopted consisted of six main girders ranging in depth from 3 ft. 9½ in. to 3 ft. 7 in. with deep curved haunches at the ends. Width of beams varied from 26 in. to 37 in. Details of bridge and method of construction are given.—JOSEPH MARIN

Design of arched bridges with fixed supports. H. CARPENTER. *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 611-23.—Object is to develop a simplified method of design for arches with fixed supports. Tables are given for calculating stresses in parabolic, open and filled spandrel arches, with different rise-span ratios and varying moment of inertia. Simplification in the calculations is obtained by designing ring to coincide with line of pressure for dead load. In this way an economical design is obtained and stresses due to arch shortening are easily computed. Expressions are derived for horizontal thrust due to dead load and arch shortening, and for bending moments due to arch shortening both at crown and springing. Values of these thrusts and moments are given in tables for different ratios of the moment of inertia at the crown and springing, and for different rise-span ratios. A table is given for each of the three types of fixed arches considered. Similar tables to the above are given for changes due to temperature, giving the values of horizontal thrust and bending moments at crown and springing produced by changes in temperature.—JOSEPH MARIN

Buildings

A new garage construction in Frankfurt a. M. (Germany). H. Craemer. *Zement* (Germany), Nov. 6, 1930, V. 19, No. 45, p. 1068-9.—Building of taxicab company is made entirely of reinforced concrete in modernistic style. Floor constructions of special design show interesting features. Details of columns, construction of windows and arrangement of storage and repair shops are illustrated.—A. E. Beitlich

An example of the mechanical solution of a statically indeterminate structure. H. W. Coultas. *Structural Eng.* (England), Oct., 1930, V. 8, No. 10, p. 344-366.—A complete analysis by means of Begg's Deformeter Apparatus is given for a six story frame two bays in width. Method of test and reduction of data for full size structure are given.—V. P. Jensen

Static consideration of concrete mantle of supports in steel skeleton structures. Leopold Berger. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1018-20.—When steel columns of large skeleton structures are provided with concrete mantles it protects them against rust and also increases supporting strength. Great savings can be obtained by decreasing dimensions of steel column and reinforcing concrete with longitudinal irons. This combination is especially applicable to supporting columns of basements and lower floors. Example is given and calculation shows savings of 45 per cent of construction costs.—A. E. Beitlich

Combining steel and concrete framing to effect economy. *Eng. News Record*, Nov. 27, 1930, V. 105, No. 22, p. 844-845.—An unusual combination of three types of structural framing in the 27-story Buckingham Building, 59 East Van Buren St., Chicago, includes built up steel columns of two different types, with both steel framing and concrete-beam framing, and also reinforced concrete columns with concrete-beam framing. Purpose of this combination was partly to effect economy and partly to keep column sizes within prescribed limits. Built up steel columns of H-section are used as far as the eleventh floor. Above the eleventh floor interior columns are also of steel, but of the Gray type, to the eighteenth floor, above which are spirally reinforced concrete columns. With columns varying in design at different heights, special methods were required for splicing the different types of steel and concrete columns. Typical details of these connections are shown. A two-way concrete floor of tile and joist type is another special feature of this building, and is used with both steel and concrete framing. Joists in both directions form a cellular construction with hollow tile filling in the rectangular spaces. Flat pieces of tile are placed under the concrete joists, being supported by dovetailed grooves in the filler tiles, so there is continuous tile surface for ceiling plastering. The floor slab over filler tiles is 2 in. thick and depth of tile is 4 in. Advantages of this type of floor construction are shallow depth and low dead load.—D. E. Larson

Rigid rectangular frame foundation for Albany Telephone Building. G. W. Glick. *Eng. News Record*, Nov. 27, 1930, V. 105, No. 22, p. 836-838.—A reinforced concrete foundation mat and two superimposed basement stories of reinforced concrete designed to act together as a rigid frame structure form interesting and unusual foundation for new section "B" of main telephone building at Albany, N. Y. This building recently completed by the New York Telephone Co. as extension to its existing plant, is 94 by 167 ft. with 3 basements and 11 stories of super-structure. Two factors made the foundation design complicated and important. These were: (1) character of the soil; and (2) necessity, because of delicate mechanism of telephone exchanges, that the building be free from unequal settlement. Building rests on clay formation over 100 ft. deep. Other buildings on this soil have settled over a long period

of time before finally coming to rest as the clay became stable under new load condition. This fact, with considerable economy in cost, led to choice of a continuous foundation mat which could distribute weight of building over its entire area. The substructure is advantageously combined into a rigid-frame, two-story-and-mat structure acting as a unit to resist unequal settlement, variations in loads and the undesirable center sag. Such a frame is virtually a truss without diagonal members and is known as the Vierendeel truss. To provide for expected settlement of building, all floors were set 1½ in. higher than those of adjoining building. Inspection of new building made in February, 1930, showed no settlement cracks in substructure or superstructure.—D. E. LARSON

Fire damage in Riga. (See PROPERTIES OF CONCRETE.)

DAMS

Stresses in gravity dams by principle of least work. B. F. JAKOBSEN. *Proc. Am. Soc. C. E.*, Sept., 1930, V. 56, No. 7, p. 1613.—Data are cited from English tests on model dams which show a non-linear stress distribution on horizontal planes. When the principle of least work was substituted for the usual assumption of a linear stress distribution in a theoretical investigation, a stress curve similar to that found in the English tests resulted. The proposed theory gives a higher bending stress at up-stream face and lower at down-stream face. It is believed that the principle of least work method will more nearly approach the true stresses than will results obtained by usual method. Belief is supported by explanations and mathematical reasoning. Previous works of Vogt, Cain, and others are cited frequently. The whole discussion applies to the masonry dam, which is usually made of concrete.—H. J. GILKEY

Circular concrete arch dam designed by simple method. R. P. V. MARQUARDSEN, *Concrete*, Nov., 1930, V. 37, No. 5, p. 33-38.—This is the second article on this method. The design formulas were stated dogmatically in October issue of *Concrete*. Present paper contains the derivations of the various formulas. The cylinder formula is used. Total compressive stress on any cross section of a hollow-cylinder wall is expressed. For purpose of analysis, the arch is assumed to be cut at the crown and to move due to radial loading, temperature and shrinkage stresses, and settlement of supports. Equations for these stresses are derived. Arch is then treated as a unit, fixed at both ends, and subjected to other stresses in addition to above mentioned ones. Bending moments, angle changes and horizontal movements are discussed and equations derived. Unit fiber stress for combined direct and bending action is derived and stated in a simple equation.—N. H. ROY

Blasting a precast dam into place. C. P. DUNN. *Civil Eng.*, Dec., 1930, V. 1, No. 3, p. 159-164.—One of most interesting and difficult features in building of a 200,000 kw. hydro-electric plant at Chute à Caron on the Saguenay River in northwestern Quebec was the diverting of swift stream by building on land a heavy reinforced concrete dam and dumping it bodily into place. Essential features involved in this scheme were (1) a fixed pier, relatively massive, to carry greater part of weight and thrust; (2) a small pier carrying part of weight, to be blasted away; (3) a cylindrical rolling face on fixed pier so designed and placed that precast dam would fall into an accurately predetermined position. Standing on end, the dam was 92 ft. high, 45 ft. thick up and downstream, and had a maximum depth of 40 ft., the bottom being contoured to fit river bottom as dam fell into place. Reinforcing consisted largely of old steel cables. Upon blasting the small pier away, the dam fell accurately into place and was not shattered by impact although a few small hairlike cracks appeared.—D. E. LARSON

The Boulder Canyon project. *Eng. Con.*, Nov., 1930, V. 37, No. 5, p. 398.—The Hoover Dam, of gravity-arch type, will rise about 727 ft. above foundation rock and raise water surface 582 ft. Length of crest will be about 950 ft., width at base 650 ft., and at top, 45 ft. River will be diverted during construction by means of coffer dam and four 50-ft. diameter concrete lined tunnels through the canyon walls. Only openings through the dam will be eight metal-lined conduits 6 ft. 9 in. in diameter for carrying the low flow of the river when the diversion tunnel plugs are being poured. The spillway will be two glory-hole type spillways discharging through 50-ft. diameter vertical shafts into the outside diversion tunnel. Tentative plans call for a power plant to develop one million horsepower. The All-American Canal will be 75 miles long and the Coachella branch 115 miles long. The dam and power plant will require 4,510,000 cu. yd. of concrete masonry. Five and one-half million bbl. of cement and nineteen million pounds of reinforcing steel will be required —N. H. ROY

The buttressed dam of uniform strength. HERMAN SCHORER. *Proc. Am. Soc. C. E.*, Nov., 1930, V. 56, No. 9, p. 1947.—The buttressed concrete dam, of which Ambursen and multiple-arch are best known illustrations, consists of three distinct members: deck, buttress, and foundation. The buttress is generally most important item as regards quantity of concrete. This has been designed on basis of vertical stress components and horizontal shearing stresses. It is proposed to substitute for buttresses a series of arched columns designed on basis of principle stresses. To eliminate trial computations, water pressure acting on an elementary deck area is considered to be transmitted to foundation in most direct manner by elementary arched columns. These elementary columns can then be combined into a monolithic buttress of uniform strength or they may be separated from one another by continuous joints to eliminate secondary stresses due to shrinkage, temperature changes, etc. Old method has sometimes been unsafe since consideration of shears on the horizontal section failed to take into account important components of stress. Proposed method offers economy and rationalizes design. The paper outlines mathematical analysis of method and gives references to detailed derivations and more complete manuscript on file in the Engineering Societies Library, New York.—H. J. GILKEY

Concrete dams in Norway. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Economical bases for stanchions—"Controvertist." *Structural Eng.* (England), Nov., 1930, V. 8, No. 11, p. 402-405.—For a steel column supported on a concrete base, the pressures on the sides of base have been included in computing the resistance of base to overturning moment due to wind pressure. Connection of column to base is considered for conditions where column is embedded in base and where column rests on top of the base.—V. P. JENSEN

Economic dimensions and shaping of slab girders of reinforced concrete under consideration of their own weight. ALPHONS CHMIELOWIEC. *Oesterr. Ing. Arch. Ver.* (Austria), Oct. 10, 1930, V. 82, No. 41-42, p. 360-1.—Work which was done by Mayer, Barck and Saliger on calculation of reinforced concrete structures is discussed. Based on mathematical considerations, a way is shown for better economic designs of slab girders in reinforced concrete construction practice.—A. E. BEITLICH

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete*, Nov., 1930, V. 37, No. 5, p. 31-32.—Chart 1 gives depth of concrete slab and area of steel necessary for balanced reinforcing. The factors employed are: span in feet, moment coefficient, live load, slab thickness and steel area. An

allowable steel stress of 20,000 lb. per sq. in., concrete stress of 1200 lb. per sq. in., and n equal to 10 are used. This chart may also be used for obtaining steel area for a slab thicker than is warranted for span and load conditions, using the steel required for the lesser conditions although in a slab thicker than necessary. This approximation is on the side of safety.—N. H. ROY

Austrian tests on concrete columns with high quality steel reinforcements. R. SALIGER. *Oesterr. Ing. Arch. Ver.* (Austria), Oct. 10, 1930, V. 82, No. 41-42, p. 361-2.—In connection with a comprehensive investigation on concrete columns with different reinforcements, conducted at laboratory for testing materials in Berlin-Dahlem (Germany) and at technical testing laboratory in Vienna (Austria), a number of column tests with high quality steels were made. Cross-section of steel varied from 4 to 14 per cent of total column cross-section. Columns were 9.8 ft. and 4 ft. long. Bearing strength was surprisingly high, and necessary cross-section of concrete column was not greater than cross-section of a riveted or welded steel column of same strength. Further tests are in preparation.—A. E. BEITLICH

Solving rectangular concrete members subjected to bending and direct stress. L. C. MAUGH. *Eng. News Record*, Nov. 6, 1930, V. 105, No. 19, p. 723-724.—The problem of designing rectangular reinforced concrete sections for bending and direct stress is now frequently encountered in use of rigid frames. In this article, author presents formulas and a diagram for design of such members. Numerical operations are made in same sequence as those for ordinary beams. This method is more direct than the methods usually presented in textbooks and the chances for numerical mistakes are less. Concrete is assumed to resist no tension and to have a constant modulus of elasticity.—D. E. LARSON

Folded plates (Faltwerke) and their application in reinforced concrete construction. H. CRAEMER. *Zement* (Germany), Oct. 16, 23, 1930, V. 19, No. 42-43, p. 990-2, 1020-2.—Author extends his studies concerning economic design and construction of reinforced concrete plates (c. f. *Zement*, July 24, 1930, V. 19, No. 30, p. 703-8; *J. Amer. Concr. Inst.*, Nov., 1930, V. 2, No. 3, Abstr. Sect. p. 72) on combination and arrangements of monolithic concrete plates junctions of which form certain angles. Analysis are given of stresses which occur in such combinations, and it is shown how this construction principle can be applied with great advantage to practice of silo and bin construction. Furthermore, consideration is given to construction of concrete roofs and multiple arrangements of this type. Writer refers to article by Ehlers in *Bauingenieur* (Germany), Feb. 21, 1930, V. 11, No. 8, p. 125-32 (*J. Amer. Concr. Inst.*, Nov., 1930, V. 2, No. 3, Abstr. Sect. p. 84) on a similar subject.—A. E. BEITLICH

Theory of similarity and models.* BENJAMIN F. GROAT. *Proc. Am. Soc. C. E.*, Oct., 1930, V. 55, No. 8, p. 1803.—Although Newton's principles of similarity and the theory of models were enunciated 250 years ago, the use of models by engineers is recent. Models often present the best form of preliminary analysis for constructing engineering works of great magnitude. The mathematical background that underlies the proper use of models is presented and attention is called to current fallacies. It is suggested that materials, including fluids should be sought, made, and arranged so as to adapt and make them available for service in models.

**Abstractor's Note:* It is deemed desirable to call the attention of the A. C. I. constituency to this paper because of the increasing use of models in connection with concrete design.—H. J. GILKEY

Concrete stadium for $6.60 per seat. *Eng. News Record*, Nov. 6, 1930, V. 105, No. 19, p. 721-722.—A concrete stadium has recently been designed for Sacramento Junior College at Sacramento, Calif., at a cost of $6.60 per

seat. By developing a design structurally independent of floorslab and using timber seats and walkways that could be put in place with a minimum of labor, cost of 22,000 seat structure was kept down to $145,000. The structural scheme consists essentially of concrete columns 14 in. square spaced 16 ft. apart both ways. These columns are connected at top by reinforced concrete girders carrying one intermediate beam each way, which reduces spans to 8 ft. With this span a 2 by 12-in. plank is ample for carrying maximum calculated load of 100 lb. per linear foot. Planks of this size were used both for seats and walkboards, the seat being stiffened by a 1 by 6-in. edge board.—D. E. LARSON

Grade separation structures untangle traffic jams. ROBERT C. KINGERY. *Concrete*, Nov., 1930, V. 37, No. 5, p. 17-19.—Four major types of structures or plans are employed in separation of intersecting highways. First type comprises two highways at different levels, with no means of turning from one highway to the other. In the second type there is an elevated structure which carries the central part of one of the highways over and across the full width of the other, all other pavements and the sidewalks on both roads remaining at grade. Third type is most desirable from the standpoint of abutting property owners. The central part of one highway is depressed, all other pavements and sidewalks remaining at grade. Fourth type is well suited to crossings where public property is available. One highway is partly depressed and other partly elevated. Diagonal lanes serve as connecting links between highways. Another effective arrangement which does not involve actual separation of grades is the traffic circle. It keeps traffic moving in both directions on both roads, although traffic on the circle itself is all in one direction, moving counter-clockwise.—C. BACHMANN

Analysis of continuous frames by distributing fixed-end moments. HARDY CROSS. *Proc. Am. Soc. C. E.*, May, 1930, V. 56, No. 5, p. 919-928.—While not restricted to concrete, the method of analysis is especially applicable to concrete construction since the method is intended to be an aid in obtaining the moments, shears and thrusts required in the design of complicated continuous frames from the reactions which can themselves be found by more or less standard means. The method is restricted by condition that joint must not be displaced but even this case can be indirectly treated. The writer emphasizes the fact that he is setting forth a *method of analysis* for the benefit of those who recognize that values of moments and shears cannot be found exactly but who desire a method of analysis that will combine reasonable precision with speed. The method is therefore one of successive approximations rather than an approximate method. Moreover it is a method rather than a treatise.—H. J. GILKEY

Small movements in concrete. W. H. GLANVILLE. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 634-43.—The paper deals with work of Building Research Station on small movements in concrete. From test data an empirical formula is obtained for the stress in reinforcing steel of concrete members due to shrinkage of concrete. Coefficients obtained from tests are given that are to be used in the empirical formula. Tests were made to determine the creep or flow of concrete under load for both plain and reinforced concrete. For plain concrete the effect of the following factors on flow of concrete was studied: (1) stress, (2) type of cement and age at loading, (3) cement content and (4) storage conditions. Tables are given for each of these factors giving the amount of creep. The tests show that (1) creep is approximately proportional to applied stress, (2) rapidity of hardening directly influences creep to considerable degree, (3) creep is approximately proportional to quantity of aggregate, and (4) influence of age of concrete at the time of loading is appreciable for about the first month.—JOSEPH MARIN

Nomogram for calculating the area of any number of bars of the same dimension. JORGE QUIJANO. *Ingenieria* (Mexico), May, 1930, V. 4, No. 5, p. 204.—A simple nomographic chart, one of a series, for use in reinforced concrete design, to compute areas of steel reinforcement. Left hand scale, dimension of bars, center scale area of bar and right hand scale number of bars, all in metric units. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating columns without spiral reinforcement (central load). JORGE QUIJANO. *Ingenieria* (Mexico), May, 1930, V. 4, No. 5, p. 209.—A simple alignment chart, one of a series, for use in reinforced concrete design, to solve formula $P = [A_c + (n - 1) A_s] f_c$. Metric units used throughout. Left hand scale A_c, middle scale P and right scale A_s for $n = 15$, $f_c = 500$ lb. per sq. in., $h/R < 40$. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating the pressure of earth (or water). JORGE QUIJANO. *Ingenieria* (Mexico), June, 1930, V. 4, No. 6, p. 227.—Alignment or nomographic chart for use in computing earth pressure for concrete retaining wall design by Rankine's formula. Explanation of use given on chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for economical design of T-beams. JORGE QUIJANO. *Ingenieria* (Mexico), June, 1930, V. 4, No. 6, p. 237.—A nomographic chart, one of a series for use in reinforced concrete design and investigation, solving the relations $d = \sqrt{\frac{r}{b^1}\frac{M}{f_s}} + \frac{t}{2}$ and $M = A_s f_s (d - t/2)$. Left hand scale $(d - t/2)$, middle scales M/f_s and A_s and right hand scale b^1/r. Explanation of use given on full size chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating the allowable distance between abutments of vertical or inclined reinforced concrete slabs as determined by diagonal tension and shear. JORGE QUIJANO. *Ingenieria* (Mexico), June, 1930, V. 4, No. 6, p. 253-254.—A nomographic chart for use in designing or investigating buttressed retaining walls which solves the relation $T = S (v - 2.8) b$. Explanation of use is given on a separate page. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating simple slabs and rectangular beams. JORGE QUIJANO. *Ingenieria* (Mexico), July, 1930, V. 4, No. 7, p. 280.—A simple alignment or nomographic chart, one of a series of such charts, for use in reinforced concrete design and investigation, solving the relations $d = 0.3558 \sqrt{M/b}$ and $A_s = 0.002532 \sqrt{Mb}$ for $n = 15$, $f_s = 18{,}000$ lb. per sq. in. and $f_c = 700$ lb. per sq. in. Left hand scale or line d, middle scales M and A_s, and right hand scale b. Explanation of use given on chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating slabs or beams with a certain degree of end restraint. JORGE QUIJANO. *Ingenieria* (Mexico), July, 1930, V. 4, No. 7, p. 291.—A simple alignment or nomographic chart, one of a series, for use in reinforced concrete design and investigation, solving the relation between A_s, d and M for $n = 15$, $f_s = 18{,}000$ lb. per sq. in. and $f_c = 700$ lb. per sq. in. Left hand scale or line L, middle scales A_s and d, and right hand scale W. Explanation of use given on chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomograph for calculating slabs or beams with a certain degree of end restraint. JORGE QUIJANO. *Ingenieria* (Mexico), Sept., 1930, V. 4, No. 9, p. 342-343.—A simple alignment chart for use in reinforced concrete design solving the relation between A_s, d and M for $n = 15$, $f_s = 20{,}000$ lb. per sq. in.

and $f_c = 800$ lb. per sq. in. Left hand scale L, middle scales A_s and d, and right hand scale W. Explanation of use given on chart and next page.—C. G. CLAIR AND M. N. CLAIR

Nomogram for determining the tension in the steel of simple T-beams. JORGE QUIJANO. *Ingenieria* (Mexico), April, 1930, V. 4, No. 4, p. 173.—A simple alignment chart, one of a series, for use in reinforced concrete design and investigation, solving the relation $M = 0.9\,dT$. Left hand scale T, center scale M and right scale d, all in metric units. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating the moment due to the weight of a slab or beam. JORGE QUIJANO. *Ingenieria* (Mexico), April, 1930, V. 4, No. 4, p. 177.—A simple alignment chart, one of a series, for use in reinforced concrete design, solving the relation $M = 0.0003\,b\,(d + e)l.^2$ Left hand scale l, center scale M/b and right scale $(d + e)$, all in metric units. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

ROADS AND PAVEMENTS

The design of a concrete mixture. J. E. MYERS AND MARK MORRIS. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Detailed description of steps involved in design of a mixture for concrete to be made of certain specific materials to have an average crushing strength of 4000 lb. per square inch at 28 days, to be of suitable consistency for pavement work, and to require the use of predetermined quantities of materials. The design was based upon methods proposed by Talbot and Richart with some variations. Previous experience with one of the materials used in this case was reported in the Seventh *Proceedings* of the Highway Research Board. Use of the designed mix on 15 miles of pavement, showed that the concrete was of satisfactory consistency, that the average strength was slightly in excess of the requirement, and that the cost of the materials used was 0.03 per cent less than the estimate.—HIGHWAY RESEARCH BOARD

Boulevard and Great Belgrano Park, planned for the City of Rosario. *Magazine Tecnico* (Argentina), June, 1930, p. 16-17.—Department of Navigation and Ports of City of Rosario, province of Buenos Aires, Argentina, proposes to construct avenue and park along Parana river. The avenue is to provide a proper view of river and the park to give proper setting for historical spot where the flag created for Belgrano first was unfurled. Two types of cross section are proposed for the avenue both occupying a total width of 164 ft. Type A cross section has two roadways 36 ft. wide, foot paths on each side of these roadways of 19 ft. 6 in. and 10 ft. 0 in. width respectively, and a central park or garden section 33 ft. wide. Type B cross section has two roadways 20 ft. 6 in. wide with footpaths of 19 ft. 6 in. on each side and a central park or garden section 45 ft. wide. Roadways are to be of reinforced concrete of 1:2:3 mix, that for the type A 7 in. thick with 9.2 lb. of steel per sq. yd. and for type B 6 in. thick with 9.2 lb. of steel per sq. yd. It is estimated that plan A would cost $1,425,-816.17 and plan B $1,087,204.60.—C. G. CLAIR AND M. N. CLAIR

FIELD CONSTRUCTION

BRIDGES

Bridges and roads of the nation (Argentina). *Magazine Tecnico* (Argentina), March, 1930, p. 15-26.—A review of progress during 1929 on various projects with details. Numerous bridges of reinforced concrete have been completed or are in the process of construction. The bridges are generally of reinforced concrete arch type with some of beam and girder. New road construction includes considerable concrete.—C. G. CLAIR AND M. N. CLAIR

Construction of Pancevo bridge over river Danube near Belgrad (Serbia). MAX HEINICKE. *Bautechnik* (Germany), June 6, 1930, V. 8, No. 24, p. 354-7.—Structure is truss bridge with 7 spans, which rest on 8 reinforced concrete piers. Approach on south side is reinforced concrete arch bridge with 5 spans. Length of truss bridge is about one mile, and of reinforced concrete arch bridge 393.7 ft. Distances between river piers from center to center are 528.2 and 531.5 ft. Spans of concrete bridge are about 83.7 ft. River piers are founded on layer of very hard clay 91.8 ft. below normal water level. Caisson method was used for their foundation; caissons are 95.1 ft. long, 41.0 ft. wide and 24.6 ft. high. Usually two piers were placed at same time, two travelling cranes being used for placing caissons and for transportation of concrete. Concrete was mixed in one-yard mixer which, together with aggregate silo and crane, was mounted on a barge. Piers and foundation slabs were made of 1:8 mix and top slabs of 1:4 mix. Parts of piers exposed to heavy ice pressures are lined with layer of natural stones. Depth of foundations of piers for reinforced concrete bridge is determined by depth of clay layer. One pier was placed using compressed air method, and two were made in open excavation with steel sheet piling of Larrsen type.—A. E. BEITLICH

The bridge over the marsh and river Warthe near Fichtwerder (Germany). KARL BERNHARD. *Bautechnik* (Germany), May 16 and 30, 1930, V. 8, No. 21-23, p. 311-5, 338-42.—Highway bridge over marsh and river consists of two parts: one steel structure of 459.3 ft. length, which crosses river at right angle and rests on three concrete river piers, and one reinforced concrete bridge over marsh, which is 1653.6 ft. long, resting on 25 piers. Concrete bridge approaches river bridge at slight angle and artificial island was built at their junction. Center opening in river is 295.3 ft. wide while two side openings are 82 ft. each. Piers of concrete bridge are placed in alternating distances of 78.1 and 82.0 ft. Roadway rests on 3 main longitudinal flat slab girders, spaced 7.4 ft. apart. Height of girders is 4.8 ft. Several drawings show details of their construction. River piers were built using compressed air method and consist of foundation 13.1 ft. wide protected against aggressive waters by brick wall, and upper shaft, 8.2 ft. wide. Foundation of piers of concrete bridge rests on concrete piles of 15.7 in. diameter. They were driven about 6 ft. into sound underground. Twelve piles support one pier. Article is accompanied by 35 illustrations which explain details of structure and portray all stages of construction.—A. E. BEITLICH

The reconstruction of the Niddatal bridge near Assenheim in Oberhessen (Germany). METZIG. *Bautechnik* (Germany), Aug. 22, 1930, V. 8, No. 36, p. 540-4.—Old railroad bridge of steel construction was rebuilt and provided with 5 reinforced concrete piers to replace 8 old steel structures. New piers were placed on new foundations. Excavations were protected by steel sheet pilings, 26.5 and 45.9 ft. long, of Larrsen type, driven from 16.4 to 23 ft. deep into sound underground. Bridge has two openings of 1490 ft. at each side and two openings of 1863 ft. between three middle piers. Foundations of piers consist of reinforced concrete slabs, 49.2 by 70.2, 43.9 by 70.2, 38.7 by 70.2 and 25.6 by 70.2 ft. Special care was required for one end pier foundation, which is directly by side and below foundation of old pier. Concreting was carried out in two sections which are tied together by heavy reinforcements. No protective coatings were necessary, since chemical analysis of groundwater showed absence of aggressive agents. Gravel was delivered by railroad cars and dumped from bridge through long wooden chutes (to prevent separation of different sizes) to mixer. Total of 157 cu. yards gravel was unloaded daily. Mixer had capacity of 17½ cu. ft. Construction period lasted five months.—A. E. BEITLICH

Concrete pits of bascule bridge resist high water pressure. *Concrete*, Nov., 1930, V. 37, No. 5, p. 13-16.—Concrete in the walls and floors of machinery pits at both abutments of the double leaf, Wabash Ave. bridge, at

Chicago resists a pressure head of 25 ft. of water. Pits comprise space formed by the four walls of the abutments. Floor is concrete slab 5 ft. thick and the entire structure acts as a unit supported on concrete sub-piers or caissons. Sub-piers are carried down 75 ft. below datum, the base of each pier being belled out. Joint between walls of pit and floor slab was double keyed and surface was roughened. Concrete in pit walls was placed in vertical stages varying from 4 ft. at bottom to 8 ft. at top. Each stage was concreted without interruption, and joints between stages were double keyed and roughened. Torpedo sand and gravel were used as fine and coarse aggregate. For sub-piers and south machinery pit, standard portland cement was used. For north pit, which was first to be concreted, Super cement was used, concrete being placed during cold weather. South pier was concreted during warmer weather. Proportions in north pit were 1:2.5:3.5, with 6 gals. water per sack of cement. For south pit and all sub-piers, mix was 1:1.7:3.2, with 6 gals. water per sack of cement. Slump in all cases was 4 in. Concrete mixing plants for pits were separate installations, because presence of river navigation made transportation across stream inadvisable. From mixing plant for north pit which included mixer, aggregate measuring hoppers for volume measurement of aggregate and water control equipment, concrete was delivered into a hopper and wheeled in buggies over runways.—C. BACHMANN

BUILDINGS

Light-concrete for construction of 400 dwelling houses in Berlin-Zehlendorf (Germany). WEISS. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1090-2.—Construction was carried out with a concrete containing gravel and Rhenish pumice stones as aggregates. Heat insulating properties make this an ideal material for small dwelling buildings. Interesting and economic methods were used for putting up formwork and distributing of concrete with a minimum of laborers. About 455 cu. yds. of concrete were placed daily. Outside of buildings was finished with white cement.—A. E. BEITLICH

Five scattered hospital buildings concreted from one plant. *Concrete*, Dec., 1930, V. 37, No. 6, p. 17-18.—On government hospital project at Northport, Long Island, N. Y., comprising six reinforced concrete buildings, concrete for five buildings was mixed at centrally located plant equipped with batchers, water-measuring device and 10-S mixer, located directly between two of larger buildings. For two buildings concrete was distributed by buggies, direct from hopper alongside main hoist tower, and above mixing plant. Hoisting tower and long chute sloped about 35 deg. with horizontal was used for building farther away. Plastic consistency was maintained by addition of gallon of admixture to each 2-sack batch. Mixing time was 1 min., 10 sec. Proportions specified and used for columns and footings and for floor slabs, beams and girders, were 1:2:4 with 6½ gal. of water per sack of cement. For roadway and plain concrete work, mix was 1:2½:5 with 7½ gal. of water per sack of cement. Mixing plant was set low as practicable and trucks hauling aggregate ran up incline built with excavated material and dumped directly into bins through open side near top of mixing plant.—C. BACHMANN

Construction of new city hall in Ruestringen (Germany). HANS KUBALL. *Bauingenieur* (Germany), May 16, 1930, V. 11, No. 20, p. 349-52.—Structure consists of three parts, separated by expansion joints; a 15-story tower structure and two side buildings. Underground conditions and presence of aggressive agents in ground water made foundation on wooden piles necessary. About 1000 piles, 12 in. and 14 in. in diameter and 40 ft. long, were driven. Great care was taken in protecting concrete against action of water. Very rich cement mortar (1:2) with addition of waterproofing compounds was used for cementing a protecting layer of bricks. Detailed description is given

of several interesting floor constructions. High early strength cement was used for tower structure and normal portland cement for construction of side buildings.—A. E. BEITLICH

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Nov., 1930, V. 37, No. 5, p. 27-29.—Details of form work around steel beams and in intervening floor panels of building having steel frame encased in concrete are shown in illustration. The entire floor form work is supported on short cross pieces 3 in. by 4 in. about 27 in. apart, held in place against the beam bottom by wire passed over steel beam and tightened by twisting above top flange of beam. Wires pass through form work at edges of beam bottom, with beam sides set against them and supported on cross-pieces. Form work for floor construction is assembled by tacking beam bottom plank to cross-pieces, then wiring this assembly to steel beams after small concrete blocks are inserted between beam bottom plank and lower flange of steel beam. Width of beam bottom must be 4 in. more than width of bottom flange of steel beam, leaving 2 in. space for concrete to pass on each side of flange. Next the lower boards of beam sides are set in place, then 2-in. planks that serve as blocking for floor joists are set on cross-pieces, and lower boards of beam sides tacked lightly against them. Floor form panels are placed and upper boards of beam sides are set and tacked lightly to ends of a few joists. Floor panels have previously been assembled by nailing ⅞-in. boards to joists. If enough men and hoisting equipment are available, a floor form panel may be made up to full area of floor panel. If lighter form units are necessary, several smaller form panels may be used instead of a large one. The ⅞-in. boards of floor panel project over beam sides. Removal of this form will be made easier if floor boards adjacent to beam are omitted from assembled panel, and set in place and lightly tacked after floor panel is set. When the form work is removed, floor panel can be lowered as soon as plank blocking is taken away. To remove floor form construction: Shore up one edge of floor form panel by temporary frame; cut wires at beam where floor panel is shored, and remove cross-pieces and beam bottom. Remove 2-in. blocking and lower boards of beam sides. Let down edge of floor panel by lowering temporary shoring frame, and remove panel and upper boards of beam sides. Ends of wires protruding from concrete may be cut off flush with the concrete unless needed to support lath and plaster. Form work for lens sidewalks consists essentially of boxes supported on ledges, beam sides being slightly battered to facilitate removal. Form work for window sill consists essentially of yokes shaped like inverted "U." Tin strap at upper right-hand corner is to provide partial hinge action so yoke can be released. Form for parapet coping involves keeping true alignment for long stretches of wall. Use continuous liners in combination with cleats and spreaders all of 3 in. by 4 in. material. Staggering liners with respect to those on opposite side of coping helps maintain alignment.—C. BACHMANN

DAMS

Construction of the locks in the river Main upstream Aschaffenburg (Germany). FRIEDEL. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1015-8.—Progress is reported of construction of 13 locks in river Main which enable navigation and serve for power production. Each lock has three openings which are 114.8 ft. wide. Ship locks are 984.3 ft. long and 39.7 ft. wide allowing passage of 3 or 4 ships at a time. On opposite side of river are located the hydro power stations. Two turbines in each station transform energy of 4,600 cu. ft. of water per second to 21 millions K. W. hours. For each station will be consumed about 80 to 90,000 cu. yd. of concrete. Locks at Obernau (Germany) are completed. Details of its dimensions and operation of gates are given. Locks at Kleinwallstadt, Klingenberg-Trennfurt and Kleinheubach are still under construction and work at Freudenberg was just started. Gradual development of these jobs is clearly illustrated.—A. E. BEITLICH

The power stations of the Lake Schluchsee. ADOLF EISENLOHR. *Bautechnik* (Germany), April 18, 1930, V. 8, No. 17, p. 259-63.—Water, which runs from Lake Schluchsee (Germany) and several other points of southern part of Black Forest (Germany) to river Rhine is utilized in three hydro power stations in Haeusern, Witznau and Waldshut. Difference between levels of Schluchsee and Rhine is 2950 ft. Meteorologic observations during 30 year period showed high average of rainfall in this region, 39.4 to 70.9 in. Steps between power stations are 659.5, 918.6 and 360.9 ft. Upper station in Haeusern is now under construction. Several small waters are collected and led to Lake Schluchsee through concrete pipe which is 5.6 ft. in diameter. Dam of gravity type, which is 108.3 ft. high and crest of which is 872.7 ft. long, dams water to level 95.1 ft. higher than original one, forming a lake of 1.9 sq. miles surface with a capacity of 141 million cu. yds. of water. Overflow is located in middle of dam. Dam is straight and has triangular cross-section. Concrete pressure line between dam and power station shows very interesting design. Inlet is located about 46 ft. below lake level. Pipe 3.7 miles long leads in nearly straight line to power station in Haeusern and has circular cross-section with inner diameter of 13.4 ft. Two gates are provided for regulation of water stream. Further description is given of construction of foundation of power house with its 4 turbines. Water basin for second hydro power station is formed by dam 495.4 ft. long, which has very favorable location. It is slightly curved and of same cross-section as first structure. Its pressure line is 1.7 miles long and 16.4 ft. in diameter. Arrangement of sites of construction, transportation and preparation of structural materials is described and illustrated.—A. E. BEITLICH

Blasting a pre-cast dam into place. (See ENGINEERING DESIGN—DAMS.)

MISCELLANEOUS

Construction cranes. "Baukrane." R. CAJAR. 1930, R. Oldenbourg, Munich and Berlin (Germany), 275 p., 354 illustrations, R. M. 18.00. Reviewed in *Z. Oesterr. Ing. Arch. Ver.* (Austria), Oct. 10, 1930, V. 82, No. 41-42, p. 367.—Book is comprehensive treatise on subject of construction cranes. Besides detailed description of different kinds of cranes, their operation and power consumption, several calculations are given for steel construction of such cranes and steel ropes.—A. E. BEITLICH

Methods of obtaining watertight concrete. WILLI SERKIN. *Betonwerk* (Germany), June 1, 1930, V. 18, No. 30.—Watertight concrete is dependent on non-absorbent aggregate, dense mortar, the sand grains of which are so graded to produce the least voids. Methods of attaining this grading are analyzed. The cement should be further made water repellent with an admixture prepared for this purpose.—H. FRAUENFELDER

Lectures on reinforced concrete construction. "Vorlesungen uber Eisenbeton." E. PROBST. 1929, Julius Springer, Berlin, V. 2, 2nd edition, 540 pp., 61 illustrations, RM. 31.50. Reviewed in *Bauingenieur* (Germany), Jan. 24, 1930, V. 11, No. 4, p. 69.—In four main chapters following subjects are discussed: reinforced concrete construction of superstructures, bridges, marine structures and miscellaneous structures. It deals mostly with practical application of different construction methods for large halls, silos, ceilings, foundations, different types of concrete bridges, sewers, pressure pipes, locks, dams, breakwaters, caissons, wells, poles and chimneys. V. 1, which appeared in 1923 illustrates theoretical side of subject.—A. E. BEITLICH

Concrete piles. M. J. MCCARTHY. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from *Conc. Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 628-32.—Methods of

casting concrete piles are given for both piles that are cast in molds before being driven into the ground and for piles cast in place. Advantages of piles cast in place over the pre-cast piles are given. Pre-cast piles are most suitable for jetties, piers, and wharves. Five methods are given for casting piles in place and the methods are compared.—JOSEPH MARIN

Coast protection. *Conc. Building Conc. Products* (England), Oct., 1930, V. 5, No. 10, p. 139.—For protection of a rocky coast with concrete, a 12 in. thickness of 1:2:4 concrete is recommended, reinforced with ½-in. dia. bars at 12 in. centers each way. The bars should project 9 in. into the rock and 9 in. into the concrete, with both ends hooked.—JOHN E. ADAMS

The concrete pump, a new concreting method. HEIDORN. *Der Bauingenieur* (Germany), May 30, 1930, V. 11, No. 22, p. 381-4.—Experiences with a new concrete pump during construction of "German House" in Flensburg (Germany) and several other construction jobs are described. Concrete was pumped 88.6 ft. high. Output was 350 cu. ft. per hr. Control tests showed, that concrete delivered from pump has great uniformity in composition and strength. Special pipe construction facilitates mounting and cleaning. Transportation and operation costs are very low.—A. E. BEITLICH

New York subway construction—IV. *Eng. News Record*, Nov. 20, 1930, V. 105, No. 21, p. 796-800.—Precast-concrete blocks are being used for lining shield-driven subway tunnels from Prospect Park to Prospect Ave. and Terrace Place in Brooklyn. The use of precast-block or monolithic concrete, at option of contractor, was specified as an alternate to cast-iron segments to determine whether a lining could be secured that would be cheaper than, but as desirable as, cast-iron lining for shield driven tunnels. On tangents the lining consists of rings of 10 standard blocks, each block 6 ft. 1 in. long on chord of outside arc, 18 in. thick and 30 in. wide, forming a tube 16 ft. 9 in. inside diameter and 19 ft. 9 in. outside diameter. Each block contains 0.8 cu. yd. of concrete and weighs 3,200 lb. Curve alignment is taken care of by tapered rings, narrowing by 3.4 in., 1⅜ in., or 2¾ in., in precisely the same manner as cast-iron taper rings. Using these three different taper rings in combination with standard rings in proper proportion gives any desired curvature of tunnel. —D. E. LARSON

New locks in Ijmuiden (Holland). ANTON VAN RINSUM. *Die Bautechnik* (Germany), May 9, 1930, V. 8, No. 20, p. 305-8.—Structure is one of greatest European ship locks. Its inner dimensions are: 1312.4 ft. long, 164.1 ft. wide and 49.2 ft. deep. Ground water level was lowered 26.2 ft. and excavation was carried 42.7 ft. below normal sea water level. Further depth of 85.3 ft. was reached by using Larssen steel sheet piling 88 ft. long driven 3 ft. in hard layer of clay. Foundation rests on concrete piles made at site of construction. A total of 6100 piles were driven. Bottom slab of lock consumed 14,130 cu. ft. of concrete and was placed in 18 hours without interruption. About 31,780 cu. ft. of concrete were placed in 48 hrs. for each side wall. Concrete mixture was very rich and made with an addition of trass as protection against sea water action. Dune sand and blast furnace slag cement without trass addition were used for part of structure. Lock heads consumed about 2½ million cu. ft. concrete of which 250,000 cu. ft. were used for concrete piles. Concrete was poured with plastic consistency, 820 lbs. of cement for 35 cu. ft. of concrete. Concrete mixer furnished 1,060 cu. ft. concrete in one hour. One distribution tower of 255 ft. high was erected.—A. E. BEITLICH

Reinforced concrete construction. Part I. Raw materials and their production. "Der Eisenbetonbau. Teil I. Rohstoffe und Herstellung." O. HENKEL. 1929, H. A. Ludwig Degener, Leipzig (Germany), 112 p., 136 illustrations, R. M. 4.00. Reviewed in *Z. Oesterr. Ing. Arch. Ver.* (Austria), Oct. 24, 1930, V. 82, No. 43-44, p. 379.—Ways and methods of reinforced

concrete investigations and research in modern institutes for testing structural materials are clearly illustrated. Concrete construction control is discussed in all details. Engines, mixers, means of transportation, storage and preparation of aggregates and cement are described.—A. E. BEITLICH

Novel shield and hexagonal concrete liner blocks features of tunnel job. *Eng. News Record*, Nov. 13, 1930, V. 105, No. 20, p. 771-773.—An electrical conduit tunnel, 4 ft. in diameter and 100 ft. long, involving the use of a new design of interlocking hexagonal precast concrete lining blocks and a shield having several innovations, has just been driven for the Philadelphia Electric Co., in North Philadelphia. Tunnel has an inside diameter of 4 ft. and block lining thickness of 5 in. One complete ring of lining contains 18 blocks, but because of their hexagonal shape 9 alternate segments are always half a block width ahead of the other nine. This feature was an essential part of shield operation. Half of the 18 shield jacks were bearing against blocks in place, while the alternate nine, retracted, were extended one at a time under low pressure to hold blocks of next half ring in place as they were set. All of segments are uniform, no key blocks being used. The blocks are elongated octahedrons, 5 in. thick (radial to center line of tunnel), 8 in. wide (longitudinal to center line of tunnel), formed by two hexagonal faces, the outer face curved to form outer diameter of tunnel, and inner face composed of three flat planes. They are fully interlocking, 3 of contact faces having grooves $\frac{9}{16}$ in. deep and other 3 having tongues $\frac{1}{2}$ in. high. Cost of this type of construction is said to compare favorably with open-cut construction up to 10 ft. deep and to be cheaper for greater depths.—D. E. LARSON

Movable distribution arrangements for cast concrete. V. MENG. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1094-6.—Modern concrete distributor which was used during construction of double locks at Kembs i. E. (France) of ship channel between Kehl and Basle (Switzerland), includes one tower located at each side of excavation and movable on 4 tracks. Height of each tower is 196.8 ft. and it can distribute concrete over a radius of 230 ft. Widths of tracks are 6.6, 59.1 and 6.6 ft. Total weight without ballast is 275 tons; traveling speed is 8.2 ft. per minute. At base of tower are two concrete mixers of $1\frac{3}{4}$ cu. yd. capacity, automatic measuring devices for water and concrete silo for 5 cu. yd. of concrete. Elevator delivers material to second silo on tower from which it is further distributed by means of pipes. Each tower can place approximately 78 cu. yd. concrete per hour, but this figure can easily be increased by installing $2\frac{1}{2}$ cu. yd. mixers.—A. E. BEITLICH

Construction of a new breakwater in the harbour of Stralsund (Germany). SEETZEN. *Bautechnik* (Germany), June 27, 1930, V. 8, No. 28, p. 435-8.—Article describes construction of three breakwaters, which are 1263.1, 1780.5 and 367.5 ft. long and replace old wooden pile structures. Greater part of new breakwaters is reinforced concrete construction. Great difficulties were encountered in finding sound underground for foundations.—A. E. BEITLICH

Ready-mixed concrete sold on strength basis at low unit cost. *Concrete*, Dec., 1930, V. 37, No. 6, p. 21-22.—Mechanical installations wherever feasible make low unit costs possible for Brooklyn plant. Cement company barge transports bulk cement in railway freight cars. Five-in. pump conveys cement to three main storage bins, from which series of screw conveyors carry cement horizontally to receiving hopper of vertical bucket elevator. Latter raises cement to 600 bbl., 2-compartment service bin, for standard and special cements, located near mixer. Self-cleaning, 300 ton aggregate bin above mixer is filled by stiff-leg derrick from nearby barges. Ten by 10-ft. hopper fitted with spout and mounted on turntable is located on top of aggregate bin. Proportioning equipment attached to aggregate bin includes quadruple, 16,000 lb. weighing batcher having four radial gates for bin and four-beam scale of

multiple type. Other attachments are 3,000-lb. weighing batcher for cement and 250-gal. water-cement ratio measuring tank. Proportioning units are adequate for 3-yd. batch. Controls and scales arranged so one man can measure cement, water, both aggregates, and discharge materials into mixer and operate mixer from operating platform. Mixer is timed for 1½ minute intervals between batches. Time required for discharge and charging is 15 seconds. Delivery trucks have 5 cu. yd. capacity, full load being made up by 3 yd. and 2 yd. batches. Field testing equipment is installed on mixer floor. Commercial laboratory representative checks control of strength, takes cylinders for tests, makes changes due to moisture variations. Concrete is usually sold on strength basis; proportions generally are 1:2:4; 1:2½:5; 1:3:6. Forty-five trucks with agitator bodies deliver up to 800 cu. yd. in 9-hour day. Aggregate bins are lined with cylinder heaters using steam under pressure of 25 to 30 lbs. Operating force includes 8 men.—C. BACHMANN

Ship locks of the Shannon power station in Ireland. FRIEDRICH WIENER. *Die Bautechnik* (Germany), May 30, 1930, V. 8, No. 23, p. 335-8.—Lock consists of two sections founded on solid rocks. Two side walls of upper section rest on 3-ft. thick reinforced concrete slab made of special cement and basalt aggregates. Bottom of lower section lies 47.4 ft. below that of upper section. Construction of it was similar. Box shaped gates of reinforced concrete are operated from bridges, spanning lock. General description of construction details and of operation of lock is given. (cf. M. Enzweiler, *Die Bautechnik* (Germany), June, 1930, V. 8, No. 25, p. 359-70, *J. of the Amer. Concr. Inst.*, Oct., 1930, V. 2, No. 2, Abstr. Sect. p. 50).—A. E. BEITLICH

New drainage pumping stations of Emscher Corporation in Essen (Germany). ALEXANDER RAMSHORN. *Bautechnik* (Germany), May 9 and 23, 1930, V. 8, No. 20-22, p. 299-303, 330-2.—Extended drainage system with great pumping station in Karnap drains area in mining district subject to slow sinking. Site of pumping station was chosen where greatest sinking is expected to take place to insure sufficient gradient at all times. Station consists of underground pumping room and engine house. Underground structure is reinforced concrete frame construction carefully made tight against ground water. Frame is a twin-cell box frame of Ehler type. Walls are divided into strips 3 ft. wide, reinforced according to load. Ground below building can sink over one third of its length without endangering structure. Excavation was made inside steel sheet pilings. Greatest care was taken in making a very dense, watertight concrete. Aggregates were carefully examined and improved by sieving and proportioning according to Fuller curve. Blast furnace slag cement was used for preparation of a stiff, plastic concrete. Excavation was first provided with layer of cinders on which was placed 6-in. layer of concrete. Brick walls were erected on all sides with a ¼-in. layer of insulating material. Then reinforced concrete floor slab and walls were erected. Description includes details of arrangement of suction and pressure pipes, engine house and adjacent building. Construction of second pumping station in Sterkrade (Germany), built in 1929, of similar design is described.—A. E. BEITLICH

Remarkable concrete and reinforced concrete structures in the steel works of Friedrich Krupp A. G. in Essen-Borbeck (Germany) FRANZ SCHLUETER. *Zement* (Germany), Oct. 2-9, 1930, V. 19, No. 40-41, p. 943-8, 968-70.—Outstanding concrete and reinforced concrete structures of greatest German steel works are described. (1) Quay at south side of main harbor has total length of 755 ft. and is made of concrete mixed 1 part blast furnace slag cement and 6 parts of gravel with an addition of trass. Precautions were taken for raising quay 6 ft., since underground is expected to sink slowly on account of undermining. Two travelling cranes for unloading boats run on top of quay and on reinforced concrete wall 781 ft. long. Forms for this wall consisted of one movable section suspended from wooden travelling crane on which was

concrete distribution equipment. (2) Storage bins for iron ore and limestone consist of one structure 361 ft. long with total capacity of 13,000 cu. yds. It is divided into 24 sections, which are 18.0 ft. long for ores and 22.1 ft. long for limestone. Partition walls are of reinforced concrete. Floors have slope of 40 and 45 degrees. (3) Foundation of one blast furnace consists of reinforced concrete slab, 55.8 by 59.1 ft. and 11.5 ft. thick. It carries a cylindrical reinforced concrete structure, inner core of which is filled with lean concrete mixture, and 4 main columns of blast furnace. (4) Chimney 230 ft. high and 9.8 ft. inner diameter at top was constructed of hollow concrete bricks, made at site of construction and provided with vertical and circular reinforcements. At height of 105 ft. above floor is located water tank around stack, with capacity of 211,000 gallons. (5) Boiler and power house are made exclusively of concrete. Furthermore, following concrete structures are described: water pipe 850 ft. long and foundation of a gas tank of 39,000 cu. yd. capacity.—A. E. BEITLICH

Bonding Concrete. *Contr. J.* (England), Oct. 1, 1930, V. 103, No. 2677, p. 1033.—Evils attendant on careless bonding of new concrete to old are not always appreciated. Shrinkage stress resistance is reduced and a poorly constructed joint may become susceptible to moisture percolation. Disintegration from frost, penetration of chemicals in solution and disfiguring efflorescence may result. Surface of well matured concrete becomes very hard and unless crust is removed by chipping, brushing with wire brush, rinsing with clean water, and a neat cement slurry added, satisfactory bond will not be formed. After slurry application, cement mortar of similar composition to that embodied in new concrete, and of plastic consistency, should be applied and followed immediately by new concrete, which should be well worked against the joint. In laboratory, excellent results have been obtained by following slurry with cement mortar, composed of two parts cement to one part sand, by weight, and following this by mortar, 2 parts sand and 1 part cement by weight. When fresh concrete is placed against this differential shrinkage is reduced to a minimum. Emphasis is laid on need for removing laitance film and layer of porous concrete immediately below it. Forms should if possible be filled to a point slightly above required height, so that poorer material may be struck off before concrete stiffens up, and within less than four hours. Too wet mixes should be avoided and when depositing under water, concrete should not be allowed to fall through water. Hacking of surface for bonding should only be done when concrete is very hard. In this case similar mortar to that in the new concrete mix should be applied in a ½ in. layer, and new concrete worked up against it. (c. f. Bonding new concrete to old. Norman Davey. *Building Research Bul. No. 9*, England).—JOHN E. ADAMS

Monolithic reinforced concrete chimneys. OTTO HOFFMANN. *Zement* (Germany), Aug. 28, 1930, V. 19, No. 35, p. 828-34.—Historical review illustrates development of construction methods for tall chimneys. Concrete form bricks with inlaid vertical and ring reinforcements were used in former years. Very little was known about stresses, in tall chimneys due to uneven heating, nor about heat permeability and radiation. Theoretical and practical experiments led to a rapid development in monolithic construction methods with adjustable, sliding forms. Investigations by Moersch, Graf, Doering, Knoblauch, Koch and Kleinlogel are cited and discussed. Use of high early strength cement made rapid concreting possible. Several outstanding reinforced concrete chimney structures, are described which show clearly progress made during last years. Chimney, 196.85 ft. high, in Obercassel (Germany), shows application of sliding forms in their early stages. System "Heine," used for construction of stack for electro power plant in Finkenheerd (Germany), shows remarkable progress in steel forms. Inner and outer forms consist of steel rings 3 ft. high, supported by scaffold on top of structure.

Structural materials are elevated through interior. Chimney is 361 ft. high and 16.2 ft. in diameter at its top. Two tallest European reinforced concrete chimneys were erected in 1929 for great power plant near Schwandorf in Bavaria (Germany). They are about 460 ft. high with inner top diameters of 24.6 and 18 ft. Structures were completed in 5 months. Stack for chemical plant was erected during severe frost periods with temperatures as low as 13° F. Entire top of structure with scaffolds was protected by closed canvas tent and was heated with steam to about 50° F. Base rests on reinforced concrete foundation slab 45.3 ft. in diameter. Total height of chimney is 262.5 ft. and its diameter at top is 11.8 ft. Thickness of walls ranges from 4.7 to 9.8 in. A very dense concrete made of high early strength blast furnace slag cement was used. Stack is lined with acid resistant bricks, 9.8 in. thick which withstand action of SO^2 containing gases which enter stack at temperature of 392° F. A chimney, which will be 426.5 ft. high when finished, is under construction now. It has a second vertical stack in its interior 131.2 ft. high serving as a flue dust collector.—A. E. BEITLICH

Completion of Moffat Tunnel of Colorado. CLIFFORD ALLEN BETTS. *Proc. Am. Soc. C. E.*, April, 1930, V. 56, No. 4, p. 679-716.—Paper supplements a former paper by R. H. Keays (cf. "Construction Methods on the Moffat Tunnel," *Trans. Am. Soc. C. E.*, 1928, V. 92, p. 63) outlining progress to August 1925. Construction started in 1923 and tunnel was opened to traffic Feb. 26, 1928. Location is 50 mi. west of Denver, at elevation about 9,200 with length 6.2 miles (longest railroad tunnel in America when constructed). It is 2,800 ft. underground at continental divide and involved 750,000 cu. yd. excavation, 16,000,000-ft. board measure timber, 2,500,000-lb. dynamite, 700,000-lb. drill steel, 700 mi. drill holes, 28,000,000-kw.-hr. electrical energy and 25,000,000 man-hours labor. The work includes a large amount of massive concrete and gunited sections. Final net cost $15,470,000. Project involves in addition to standard 16 by 24 ft. railroad tunnel an 8 by 8-ft. pioneer bore from which the excavation at several different headings could proceed from 8 by 8-ft. cross-cuts at about 1,500 ft. intervals. The pioneer bore is to be used as a water tunnel to augment Denver's water supply. West 2.5 mi. was through soft ground and heavy timbering, steel reinforcement, concrete lining, and development of "Lewis Traveling Cantilever Girder" were among important engineering features involved. Surveys and closures, housing facilities, geological and general engineering features are covered in the paper. Concrete mixers and guns for the job cost $26,000. Aggregates were crushed and screened from the tough granite excavated from eastern half of the tunnel. Some stretches of tunnel subject to air slacking were lined with a 1½ 3-in. layer of gunite (1:3 proportions) at about one tenth the cost of timbering. This proved very satisfactory when there was no weight to support. In bad ground very heavy concrete lining was required. Mix was 1:2:3 (water-cement ratio 0.8 to 1.0). Special reground cement was used to give strengths of 2,000 and 4,500 lb. per sq. in. in 48 hours and 7 days respectively. Reinforcement of 30-lb. rails and 1-in. bars was placed between 5½-ton steel sets. Folding wooden forms in 4 ft. sections were used for invert. Then steel forms mounted on a timber jumbo carriage were moved along tracks on finished invert and jacked and bolted into position. Side and top concrete was fed from concrete guns through 8-in. steel pipe to crown of arch. Flowing into place was assisted by workmen with shovels and one-man air hammers used outside the forms. Mixing time was 1½ min.; gun pressure 80 to 110 lb. per sq. in. and a slump of 6 to 7 in. at mixer was necessary to obtain a 4-in. slump in the form. Both stationary and moving gun setups were used and inspection windows were provided in forms. There was segregation in pipe lines if more than 100 ft. long. Concrete thickness ranged from 1 to 5 ft. and a total of 916-lin. ft. of steel lining was buried in 12,000 cu. yd. of concrete including that poured around wooden sets. Provision has been made for placing some added concrete lining each year if necessary. Concrete lining cost $270 per ft. (8.5 cu. yd. at $31.74). A length

of 1,485-ft. of the pioneer water tunnel (62 ft. away) was lined with corrugated pressed steel liner plates which were covered with 4-in. of gunite. This supported loads estimated at 129,000 lb. per lin. ft.—H. J. GILKEY

ROADS AND PAVEMENTS

Maintenance costs. H. K. BISHOP. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Description of some of the ramifications involved in attempting to compare maintenance costs in different jurisdictions, such as quality of maintenance, accounting methods, equipment, traffic, width and climate.—HIGHWAY RESEARCH BOARD

Repair of pavement settlement. W. H. ROOT. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—A description of a unique method for raising sections of pavement that have settled, by pumping mud through holes in the slabs, with operating and cost data.—HIGHWAY RESEARCH BOARD

Fillers and cushion courses for brick and block pavements. J. S. CRANDELL. *Summaries Papers Reports*, Highway Research Board, Dec., 1930. —The ideal filler to meet the conditions set forth in the report has not yet been found. Some experiments that are under way but not yet ready for study are mentioned.—HIGHWAY RESEARCH BOARD

Recent developments in construction methods and appliances. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—An illustrated discussion showing the latest types of all kinds of equipment peculiar to road construction, and tracing relationships between improved road work, lowered costs and machinery development. Improvements have been brought about principally through (1) changes made necessary by reason of modifications in construction practices, (2) the adaption of existing devices to new uses, and (3) refinements and improvements made primarily for the purpose of increasing operating efficiency. Attention is also called to some further improvements, the need for which is apparent.—HIGHWAY RESEARCH BOARD

Removal of ice from pavements. B. C. TINEY. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Attention is called to the need for an investigation to develop information relative to the most effective and economical materials and methods for use in dealing with icy road surfaces.—HIGHWAY RESEARCH BOARD

Economies of grading and paving. A. STELLHORN. *Contr. J.* (England), Oct. 22, 1930, V. 103, No. 2680, p. 1268.—An excess thickness of slab is regarded as evidence of poor engineering supervision since it may represent considerable expense in a large job. It is suggested that screed of finishing machine and sub-grader be set absolutely dependent on each other, a section of grade being subgraded and checked for thickness by running finishing machine in front of mixer and testing grade. The extra work is much less in value than the cost of concrete saved on use of scratch board, which usually causes a grade to be $\frac{1}{8}$ to $\frac{3}{8}$ in. low. Some notes are given on the method of dealing with the excavation, haulage and refilling, rolling and finishing of grades, and then on the paving operations.—JOHN E. ADAMS

Systemization of Vercellese Street in the Province of Milano. ERIK SILVA. *Le Strade* (Italy), March, 1930, p. 79-84.—Development of provincial streets radiating from city of Milano is most important part of a general plan of street systemization which has been progressing since 1927, and includes not only placing of new road surfaces, but also of rearrangement of the gas, electric, sewer, telephone, power and car lines. Vercelles St., an important artery is 82 ft. wide constructed, after much study, with a central safety zone

23 ft., two thoroughfares of 19.7 ft. and two side walks of 9.8 ft. wide. The street sections are given a slope of 2 per cent to the right side for drainage and to give easier driving. The base of the road is concrete and the surface asphalt. On a section of 2390 sq. yd. "Walterite," a penetration type of bituminous surface, was used. Every effort was made to plan the road so that subsequent breaking through to change piping, etc., would not be necessary. The cost was about $24.50 per sq. yd. of which $10.70 was for paving.—C. G. CLAIR AND M. N. CLAIR

Modern concrete pavements. E. K. SMITH. *Canadian Eng.*, Sept. 9, 1930, V. 59, No. 11, p. 309.—Concrete pavements in cities built 36 years ago are still giving good service. In the United States 50,000,000 sq. yd. of concrete pavements are built yearly. Materials are tested before use and proper mixture is determined by laboratory studies. Some methods which have resulted in economy with increase in quality are proportioning aggregates by weight, central mixing plants with delivery to job in some type of agitator body, greater attention to limitation of water content of batches, rigid steel side forms which assist in obtaining a smooth surface free from waves or inequalities, improved finishing machines which permit the use of batches of low water content, greater attention to curing by means of "ponding" or the application of some surface coating which prevents evaporation, and proper placing of expansion joints properly protected by a mastic filler.—G. M. WILLIAMS

Seattle's concrete pavements cured with continuous sprinkler. H. F. FAULKNER. *Concrete*, Dec., 1930, V. 37, No. 6, p. 19-20.—City's specifications require that as soon as pavement can be sprinkled without injury to fresh concrete, it shall be covered with burlap sheets two feet wider than pavement, the contractor supplying sufficient burlap to cover day's run, with 2 ft. lap for each sheet. Covering must be kept continuously wet and maintained in place until system of continuous sprinklers is installed following morning. Sprinklers shall be operated day and night without interruption for 10 days. Perfected sprinkler head throws very fine spray or mist, and when screwed into tees on 1¼-in. pipeline about every 20 ft. will continuously water 25-ft. pavement. Automatic, intermittent valve control in hydrant lead supplying line of sprinklers is also permitted. Device may be adjusted to give on-and-off periods of sprinkling, saving 75 per cent of water. To overcome strong wind, line of heads is tipped toward wind.—C. BACHMANN

A section of pavement with rigid reinforcement. ERIK SILVA. *Le Strade* (Italy), Feb., 1930, p. 58-61.—Aurelio Saffi Street in Milano was paved in 1910 with compressed asphalt and has stood up well under traffic except for portion occupied by two car tracks. Street was repaired by excluding traffic and placing a rigid reinforced concrete base 12 in. thick under the rails. Rails are attached to short sections of cross beams spaced a foot or two apart and embedded in the concrete. Machine cut blocks of porphyry quartz with bituminous mortar joints were used for wearing surface. A sand cushion is used between concrete base and blocks. Street was opened to traffic after one month. The economics of this type of pavement relative to the less rigid types is discussed.—C. G. CLAIR AND M. N. CLAIR

Patching concrete roads. GEREKE. *Betonstrasse* (Germany), July, 1930, V. 5, No. 6.—Tests of repairing concrete roads on a test street in Braunschweig showed good results. One part (by volume) cement, 1.6 parts sharp sand, 2.4 parts granite chips, ¼ in. to 1 in., mixed with 1 part of Sika No. 3 and 1.5 parts water were used. Holes were first washed with a mix of one part sand and one part cement. Material was applied in 6 to 7 minutes. Tests were made after 45 hours and showed different results for different kinds of cement used. It was estimated that according to test results the patched places could be opened for traffic after 20 to 24 hr.—R. J. DIETRICH

Concrete pavements. *Magazine Tecnico* (Argentina), April, 1930, p. 23.—A general statement of advantages of concrete pavements as found from their use in Argentina. The status of concrete road construction in the Province of Buenos Aires, Argentina, as of September 1929, is given as:

	Sq. Yd.	Cost Per Sq. Yd.
Constructed	125,800	$15.20
In Construction	103,800	11.10
Authorized	210,000	10.70
Projects ready	70,500	13.45
Projects being studied	478,000	11.50

This gives a total of 990,000 square yards at a general average price of $11.90 per sq. yd.—C. G. CLAIR AND M. N. CLAIR

SHOP MANUFACTURE

Observations on cast stone and the cast stone industry. *Betonwerk*, (Germany), April 6, 1930, V. 18, No. 21.—The proper name for this product is discussed. "Betonwerkstein" appears most popular. The product and its qualifications are described.—H. FRAUENFELDER

Concrete structural members and concrete products in slab construction. R. ROLL. *Beton Eisen* (Germany), Oct. 20, 1930, V. 29, No. 20, p. 370-373.—A brief summary is given of types of precast reinforced concrete load-carrying members and of concrete blocks for use in slab construction. Details of the various systems are shown in the figures. The article presents a view of the present stage of this field in Germany, and companies engaged in the manufacture of these products are listed.—A. A. BRIELMAIER

Profitable manufacture of concrete building units. FRED A. SAGER. *Concrete*, Dec., 1930, V. 37, No. 6, p. 33.—Business of producing and selling concrete products exhibits widest range in methods adopted and used by various concerns engaged in such manufacture. If, by keeping simple records, small plant can change manufacturing schedules from a haphazard to an orderly one with resulting saving, course should be adopted. Fundamentals of good plant design should be known and, if not embodied in present layout, worked into it. Most suitable equipment should be used, though in some cases this may not mean the most efficient machine. This series of articles will analyze business of moderate sized plant and arrive at certain fundamental conclusions as to stock of all types to be carried; plant required; general features of economic plant design and use of plant land; use of simple records to interpret costs; matters of deliveries and contact with customer during delivery; and markets and sales.—C. BACHMANN

Concrete units win way into large buildings. RUSSELL ACKER. *Concrete*, Dec., 1930, V. 37, No. 6, p. 13-16.—In bidding for Fort Wayne, Indiana, masonry market on quality basis, efficient plant having large production volume was found necessary and equipment installed accordingly. Outstanding unit is automatic Anchor junior stripper with tamper, power feeder and power strikeoff, having daily capacity of 2,500 standard size units. Large variety of sizes and quick mold changes are possible. Brick-making attachment added recently produces brick economically with low capital investment. Other units are a Hobbs downface machine under an Anchor floor tamper, with a daily capacity of 500 faced, standard size units, and Anchor tamper over a stripper for making two light-weight concrete roofing slabs at one operation. Masonry units are transported to one of three kilns on cars and steam-cured not less than 12 hours, then moved to yard on transfer track. Four men, one at mixer, one machine man and off-bearer, one all-around man, and one on truck delivery, are employed. Three 1-ton trucks comprise delivery equipment. Calls on architects and other logical prospects met with little success until

samples were left at offices and use of concrete as partition units in large eastern and southern buildings cited. Laboratory tests, building code committee recognition of units, and finally inspections of the plant, with high-grade equipment producing clean-cut block, neatly stored, converted architects. Largest order has been for 100,000 units; others for 15,000 to 20,000.—C. BACHMANN

Concrete cable covers. *Conc. Building Conc. Products* (England), Oct., 1930, V. 5, No. 10, p. 133.—To replace earlier clay product for protection of underground cables, many firms are manufacturing concrete cable covers, which are less easily broken, and which are provided with interlocking devices. A description is given of the latest type of cover, which is 12 in. by 6 in. by 1½ or 2 in. thick and the ends of which are concave and convex. To ensure that these ends shall remain tightly together, covers are provided with loops of galvanized iron wire projecting at each end, the loop at concave end being passed through loop at convex end of adjacent cover, and turned over with the fingers. These wires pass through the cover and act as reinforcement as well as tie, and serve to hold the pieces together in event of their being struck by a pick. Tops of the covers are stamped "electricity." They are composed of clean shingle, and sand from ⅜ in. down mixed with portland cement in the proportions of 1:2:4 and the materials are consolidated in moulds by vibration. Other products such as curb sections and flags are described, with illustrations of plant.—JOHN E. ADAMS

New method of stone preservation. *Betonwerk* (Germany), Oct. 26, 1930, V. 18, No. 43.—It is stated that each kind of stone and cast stone requires special treatment for its preservation. The treatment must not prevent breathing of stone. Experiments have been conducted with Silicic acid dissolved in alcohol or benzol as a surface treatment. This surface treatment hydrolizes slowly and cements grains together. Research being continued.—H. FRAUENFELDER

Tightening the requirements for making pipe C. M. HOWARD, *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 20-21.—In plotting results of a number of sieve analyses it was noticeable that material passing through No. 8 sieve was quite uniform, so this sieve was taken as basis for comparison. Samples were broken on No. 8 screen and percentage of each sample retained was noted. Then the portion passing No. 8 was screened and results plotted. Uniformity of material below screen was quite startling, as was varying percentage of total sample passing. The result is quite obvious. A great deal of variation in material can be eliminated by breaking on No. 8 screen. The aggregate should be controlled by using a maximum of ⅜-in. gravel in all thin-shelled pipe; and it is believed that this is a maximum to be used in any machine-tamped pipe regardless of the thickness, because tightness of compacting is a matter of how large a rock the tamp stick will bed into the dry mix required, and necessity for compactness is as great in one size of pipe as in another. Tamp stick clamp has to be tighter in proportion to effective end area of tamp stick. In large size pipe the stick is larger, friction on stick is set tighter, and there is just that much more chance that tamp stick will have to bed two or three of maximum size aggregate particles in same blow.—E. S. HANSON

MATERIALS

AGGREGATES

Sand and gravel. "Sand und Kies." E. TRUEMPENER. 1930, Kalkverlag, Berlin (Germany), R. M. 4.20. Reviewed in *Zement* (Germany), Nov. 27, 1930, V. 19, No. 48, p. 1146.—Book deals with application of sand and gravel in concrete and other industries and describes properties, which are necessary for such purposes. Modern practice demands higher standards of purity and uniformity. A comprehensive list of references and many illustrations are given.—A. E. BEITLICH

The sand and gravel industry and its problems—Part II. JOHN ZOLLINGER. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 56.—Dredging pump described has special protection by pressure water which protects both runner and shell. This has decreased repair bill two-thirds, based on two years experience. Dredge with portable hull and an 8-in. pump—has hull made of four 8 x 16-ft. pontoons. Four men can easily handle and unload one pontoon. The cabin is in ten sections. The advantages of portable type are lower expenses of transportation and launching and a greater resale value.—EDMUND SHAW

Glacial sand-gravel aggregate found suitable for paving work. L. H. KOENITZER. *Concrete*, Jan., 1931, V. 38, No. 1, p. 29.—Glacial material available in eastern Nebraska and portions of Kansas has maximum size of 1½ in. Most material passes ¾-in. mesh sieve and has fineness modulus of 4.10 to 4.50. Study was made on over 450 9-in. spheres for wear tests, 140 6 by 6 by 35-in. beams for flexure tests, and 500 3 by 6-in. cylinders for compression tests, to determine range of possibilities of the aggregate. Data show that for variable mixes, with constant water-cement ratios and fineness moduli of 3.50 and 4.50, mixes containing 20 per cent passing 28-mesh sieve gave higher strength values than similar mixes containing 10 and 15 per cent passing same sieve; increase in water-cement ratio decreased strength and increased per cent of wear. Mixes giving greatest strength are listed. Total aggregate on six miles of paving showed average modulus of rupture greater than 560 lb. per sq. in. at 10 days; compressive strength of 3800 lb. per sq. in. at 28 days. Total aggregate satisfactory when precautions are taken for expanding and contracting of concrete.—C. BACHMANN

New England quarry plant makes periodical changes. *Rock Products*, Nov. 22, 1930, V. 33, No. 24, p. 41-46.—At beginning of season just passed, the Middlefield, Conn., plant of the Connecticut Quarries Co. changed from steam to electric power and from rotary to vibrating screens beside making other improvements reviewed. Power change was made by installing a single 500-h. p. synchronous motor to drive principal crushers and screens. High and uniform power factor resulting has caused a material saving in power cost. Vibrating screens have shown lower power and general maintenance costs and a higher cost for screen fabric, but are found on the whole to be more economical to operate than rotary screens. Quarry face of hard trap rock is 1,500 ft. long and from 80 ft. to 190 ft. high. It is worked by a combination of coyote holes and snake holes, each shot being carefully designed on the drawing board. The maximum burden allowed is 60 ft. Loads as great as 54,000 lb. of 40 per cent powder have been shot. The recovery is estimated to run between 5 and 5½ tons of rock per lb. of powder and practically no secondary shooting is required. Haulage is by motor truck.—EDMUND SHAW

A year's operating experience with a new crushing plant. NELSON SEVERINGHAUS. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 33-38.—The superintendent of Consolidated Quarries Corp., Lithonia, Ga., tells what a year's experience with a new plant has taught. Blast hole drilling originally

was 18 ft. spacing with 20 ft. burden. The rock broke clean from the face but fragmentation was poor requiring too much expensive block-holing. The burden was reduced to 16 ft. for the second shot with much better fragmentation. Drilling is now being tried with 5⅝-in. holes set 18 x 20 ft. The charge will build up faster in this size hole and less stemming will be required. It cost 79¢ to break the average oversize piece by mudcapping and 34¢ by blockholing. As the quarry is in a locality where the same rock is cut for building stone and paving blocks, the method of breaking by short holes and wedges was tried. Cost was about same as for blockholing but larger pieces were left to be handled by shovel. Reversable control on crusher motor has saved cleaning out crusher after shut down. Author believes lowering crusher speed from 170 to 125 r. p. m. would reduce peak load, lessen wear on bearings and still give sufficient capacity. Due to accumulation of fines in bins and stockpiles, recleaning vibrating screens have been put in over loading bins. Some washed sand is being produced from screenings by the use of an Anaconda type rising current classifier.—EDMUND SHAW

Effect of flat and elongated pieces in crushed stone used as an aggregate in concrete pavements. A. T. GOLDBECK. *Crushed Stone J.*, May, 1930, V. 6, No. 5, p. 2-11.—This investigation was designed to answer two questions: (1) Are flat pieces actually harmful in finishing of concrete pavements? (2) What effect do flat pieces have on strength of concrete? Flat fragments were defined as those whose length exceeds five times the least average dimension. Only larger size flat pieces, those retained on a 1¼-in. screen, were considered harmful. Slabs 3 ft. wide, 6 ft. long and 6 in. thick were made of 1:2:3½ concrete with 0, 5, 10, and 15 per cent of flat pieces, and of 1:2:4 concrete with 0, 5, and 10 per cent of flat pieces. Two slabs of 1:2:4 concrete were made in which gradation of stone was varied. Best index of difficulty of finishing was considered appearance of concrete after each successive passage of steel screed, and finally, after use of wood float. Slabs were broken into beams which were tested at age of 28 days for modulus of rupture. Following conclusions were drawn: (1) Presence of up to 10 per cent of flat pieces in stone has no noticeable effect on difficulty of finishing 1:2:4 concrete, provided same stone with flat pieces removed produces concrete which may be finished without difficulty. (2) The presence of flat pieces only slightly increases percentage of voids in stone and requires no change in water-ratio to produce concrete of same consistency as obtained with stone having zero per cent of flat pieces. (3) Changes in gradation of stone have a far greater effect on percentage of voids and mortar voids ratio than flat pieces. (4) The 1:2:3½ mixes were more workable than 1:2:4 mixes. They have much greater factor of safety against variables which may occur in aggregates than 1:2:4 mixes. (5) Flat pieces up to 10 per cent in 1:2:4 mixes and 15 per cent in 1:2:3½ mixes do not decrease strength of concrete. (6) Indications are that flat pieces do not lie in a position such as to cause trouble at surface of a concrete slab.—J. E. GRAY

CEMENT

Investigation of portland cements, rich in ferric oxide—II and III. SHOICHIRO NAGAI AND K. AKIYAMA. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), 1930, V. 33, p. 286-90, 421-8. Reviewed in *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1096.—Cements were burned between 2400 and 2460° F. with a ferric oxide content of 7½ to 11 per cent. Raw materials were limestone, puzzolana, copper slag and clay. Mortar strength (1:3) was tested by means of small-piece testing method, invented by author. Compressive strength is lowered when ratio $CaO:SiO_2$ is decreased.—A. E. BEITLICH

Movements of materials in rotary kiln and power consumption. A. NILSSON. *Zement* (Germany), Nov. 20, 1930, V. 19, No. 47, p. 1105.—Mathematical study of movement of materials in rotary kilns. Power consumption

can be calculated by means of formula derived by author. Factors taken into consideration are: weight of materials in kiln, number of revolutions of kiln, changes in weight of mix due to calcination and drying and internal friction of materials. Formula is not considered for use in mill practice.—A. E. BEITLICH

Dusting of portland cement clinker. KATSUZO KOYANAGA. *Rock Products*, Nov. 22, 1930, V. 33, No. 24, p. 66-68.—The opinion of Dyckerhoff is quoted, that dusting of clinker comes with a change of dicalcium silicate from the β to the γ modification which is accompanied with about 10 per cent increase in volume. The author then describes his investigation of dusting clinker which was made through the accidental inclusion of a piece of flint stone, probably, with a raw meal of known composition. Microscopic examination showed the original clinker to be largely Alit. The dusting portion of clinker was mainly Belit. It was much higher in silica and lower in lime than original clinker. Tests were made by fusing pure reagents to $CaO.SiO_2$, 2 $CaO.SiO_2$, and 3 $CaO.SiO_2$. The melt of 2 $CaO.SiO_2$ broke into white powder. Tests were then made with two kinds of raw meal, one high and the other low in sesquioxide. Silica, some of which was taken from such bricks as are used in kiln linings, was added and rods were molded which were melted in an oxy-acetylene flame, allowed to drop in a heated crucible and then cooled. Six series of these melted clinkers were plotted on a ternary diagram. The line connecting the dusting points was nearly straight through the points of 2 $CaO.SiO_2$ mixtures. When the dusting clinkers were tabulated dusting was shown to occur through a wide range of chemical composition. But the contents of lime and silica can never be higher than 64.92 per cent CaO or 35.08 per cent SiO_2. The higher the content of sesquioxide lower the content of lime and silica and the longer time it will take to begin dusting. It is thought that dusting more often occurs with silica than aluminous brick lining.—EDMUND SHAW

Investigations on operation and efficiency of air separators. II. P. ROSIN AND E. RAMMLER. *Zement* (Germany), Oct. 16, 23, 30, Nov. 6, 1930, V. 19, No. 42, 43, 44 and 45, p. 984-8, 1011-4, 1035-8, 1060-1.—An illustrative case is worked to analyze process of grinding in tube mill with an air separator. Tests were conducted with coal. Amount of feed was varied while all other conditions were kept constant. Samples were taken at various places and their fineness determined. Influence of rate of feed and number of revolutions of air separator fan on (1) fineness of end product, (2) number of circulations of feed, (3) load of elevator and actual load of mill, (4) efficiency of air separator and (5) power consumption were carefully examined. Results of tests were used for expression of encountered conditions by mathematical formulas. Use of these formulas and their application to mill practice are especially emphasized. Reference is made to work by Rosin and Rammler, *Zement* (Germany), June 27, July 18, Aug. 1 and 8, 1929, V. 18, No. 26, 29, 31 and 32; A. B. Helbig, *Ibid*, Jan. 9, 1930, V. 19, No. 2, p. 25-8, *J. Am. Concr. Inst.*, April, 1930, V. 1, No. 6, Abstr. Sect. p. 69.—A. E. BEITLICH

Reply to the article by Alton J. Blank on ignition losses. KATSUZO KOYANAGI. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 67.—Discussing Mr. Blank's view that the harder clinker is burned the less moisture it will absorb, the author says moisture absorbed in storage is function of two factors, (1) quantity of moisture in atmosphere and (2) hardness of clinker. But a higher burned clinker may absorb more moisture than a soft burned clinker if it is stored in atmosphere with more moisture. Author's experience is quite opposite to Blank's statement that he has never known strength qualities of cement to be bettered by storage whether in humid or dry atmosphere. Hourly samples of cement are taken at outlet of mills and packers and mean strength found. Mean strength after one month of storage is always a little higher than that of same fresh ground cement. Many tests on effects of storage have shown

that storage of moderate length is effective in increasing strength of cement, or at least it is not harmful.—EDMUND SHAW

Investigation of portland cement and its constituents by means of measurements of vapor pressure. F. KRAUSS AND G. JOERNS. *Zement* (Germany), Nov. 6, 1930, V. 19, No. 45, p. 1054-5.—New method is proposed, which enables investigator to study formation of hydrates and to observe kind of hydrates which are present. Nature of process (if it is formation of crystals or colloids) and amount of water which is taken up can be studied. Two different ways are worked out, one is determination under isothermal conditions and one under isobar conditions. Latter way is superior for study of portland cement. Substance is exposed to vacuum at certain temperatures. Vapor pressure will always be constant when true hydrate is present until this is decomposed, which fact will set off state of equilibrium. Micro-tension-eudiometer by Krauss and Schriever and its operation is described.—A. E. BEITLICH

Reaction of carbon dioxide and acetic acid on portland cement. HANS TH. BUCHERER AND F. W. MEIER. *Zement* (Germany), Nov. 27, 1930, V. 19, No. 48, p. 1134-8.—Suspensions of cement in water were treated with carbon dioxide and amounts of lime and silica, which went into solution, were determined. Intention of investigators was to remove lime as completely as possible without attacking silica, but it was found that besides calcium bicarbonate considerable silica was dissolved. Acetic acid was then used in 1 per cent solution in addition to carbon dioxide with specially prepared indicator. Carbon dioxide was introduced under constant stirring and calculated amount of 1 per cent acetic acid was added drop by drop avoiding any excess acid. About 24 per cent of total silica was dissolved. Further tests were carried out with acetic acid alone and phenolphthalein as indicator. Results of tests are plotted in curves and given in tables. Comparisons were made between reactions of great number of different cements (portland and high alumina) with acetic acid and their strength properties. It was found that cements with low compressive strength show steep titration curves with only small consumption of acetic acid while cement with similar chemical composition but high compressive strength consume more acid during titration and have flat curves.—A. E. BEITLICH

Researches on the rotary kiln in cement manufacture. Part VI. GEOFFREY MARTIN. *Rock Products*, Oct. 25, 1930, V. 33, No. 22, p. 43-44.—Heat radiated from kiln walls in decarbonating zone is 43,338 S B. t. u. in 1 hr., where S equals the surface in feet. To make 1 lb. of clinker, decomposing 1.1905 lb. $CaCO_3$ at 805° C., 811.92 B. t. u. are required. Heating 1-lb. material from 805° C. to 100° C. requires 148.7 B. t. u. The total heat per lb. of clinker which is required in the decarbonating zone is therefore 148.7 plus 811.9 = 960.6 B. t. u. And for one ton of 2240 lb. it is 2.152×10^6 B. t. u. The coal required is shown to be $2240 \frac{M}{y}$ lb. per hr. which evolves $28.23 \times 10^6 \frac{M}{y}$ B. t. u. per hr. Unbalanced residue of heat must be counterbalanced by same amount of heat radiated from kiln walls, because the temperature of walls must steadily increase as heat plays upon them until they radiate back as much heat as they receive. Equating the heat radiated away against the free heat from the coal gives: $43358\ S = M \times 10^7 \left(\frac{28.23}{y} - 2.152\right)$ which calculates to $S = M \left(\frac{651}{y} - 49.63\right)$ If y is assumed to be 3 then S equals 167.4 M. A table is appended giving the surface in the decarbonating zone required for outputs of 1 to 20 tons per hr., based on this value of y. This formula is said to be new. The length of the decarbonating zone is figured from

surface and diameter. It has been previously shown that $d = 2.98\sqrt{M}$ and as $S = \pi\, dL$ then $L = 17.88\sqrt{M}$. A table gives the required lengths of decarbonating zones for outputs of 1 to 45 tons per hr. and diameters from 3 ft. to 20 ft.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part VII. GEOFFREY MARTIN. *Rock Products*, Nov. 8, 1930, V. 33, No. 23, p. 36.—Theoretical deductions of sizes of preheating and drying zones of kilns are difficult. Empirical experiment has shown following formulas to give approximately correct results for wet process kilns, slurry to contain 40 per cent moisture, where L is the length of the kiln: Cooler end, 2/21 L; Clinkering zone, 4/21 L; Expulsion zone, 5/21 L; Preheating zone, 3/21 L; Drying zone, 7/21 L. The relation of length L to diameter d is worked out to: $L = 25\ (d - 1.5) + 0.2\ (d - 1.5)^2$. For dry process kilns the relation is: $L = 20\ (d - 1.5) + 0.2\ (d - 1.5)^2$. d is stated to be diameter of iron shell in the decarbonating and drying zones. Where diameters vary up and down the kiln d must be taken as the average diameter. Commenting on this article, the Editor, *Rock Products* says: "Apparently about 260 to 365 ft. is the proper length for the usual American wet process kiln, 11 ft. 3 in. diameter of the steel shell. For the dry process the kiln should be only about 50 ft. shorter, which would seem to prove most of our dry kilns too short."—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part VIII. GEOFFREY MARTIN. *Rock Products*, Nov. 22, 1930, V. 33, No. 24, p. 47-49.—Theory of expulsion of CO_2 from lime and magnesian carbonates is discussed with an explanation of disassociation pressures and Johnstone's table. It is concluded that in a rotary cement kiln CO_2 begins to evolve at 805° C. at the point where the raw material enters the calcining zone. But as it enters the sintering zone the temperature at which it begins to evolve CO_2 is reduced to 748° C. Practically this means that evolution of CO_2 is almost infinitely slow at these temperatures at the points in the kiln mentioned but as the temperature of the raw material increases beyond these limits the rapidity of expulsion rapidly increases. At 900° C. the bulk of the CO_2 has probably been expelled, last traces going off probably at 1100° C. The time factor is of great importance. Years would be required at 805° C. at start of calcining zone whereas a few minutes would suffice if raw material temperature were brought to 900° C. It would be very desirable to determine minimum time of passage through furnace at temperatures from 800° C. to 1100° C. to arrive at time constant for different temperatures. Influence of speed of kiln gases over surface of material should also be investigated. Having collected such data one would be in a position to design a kiln on scientific lines. At present kilns are designed on rules founded on experience for lack of chemical and physical data. An appendix to this part gives a digest of references to disassociation of carbonates published in literature relating to cement.—EDMUND SHAW

Indirect determination of lime in cement and similar materials with standard sodium oxalate (Sorensen). A. HEISER. *Zement* (Germany), Dec. 4, 1930, V. 19, No. 49, p. 1154-5.—Titrometric method for determination of lime is described. Latter is precipitated with a known excess of sodium oxalate which in turn is back-titrated with solution of potassium permanganate of known concentration. Sodium oxalate of highest purity is used for this purpose. Potassium permanganate solution is boiled and filtered before standardization. Procedure of determination is as follows: Filtrate from R_2O_3 precipitate (of 1.0000 g. sample) is heated near boiling temperature, made acid with strong HCl and then 1.9832 g. sodium oxalate is added. After boiling for about 5 minutes more HCl is added until precipitate is dissolved. Calcium oxalate is then precipitated by adding slowly strong ammonia to slight excess. After settling, solution is filtered into 500 cc. measuring flask and carefully washed with hot water. After cooling flask is filled with water

to mark and 100 cc. are withdrawn with a pipette. To this latter portion are added 400 cc. water, 20 cc. strong sulphuric acid and 20 cc. manganese sulphate solution. After heating to 158° F. it is titrated with 1/10 normal potassium permanganate solution. Volume corrections are necessary when temperature in laboratory changes frequently. Method is simple and gives good results.—A. E. BEITLICH

Marquette has new tow boat. *Rock Products*, Dec. 6, 1930, V. 33, No. 5, p. 68.—The "William Dickinson," 125 ft. long, 26-ft. beam and 7-ft. molded depth, new tow boat of Marquette Cement Manufacturing Co., tows four self-unloading barges, each of 5000 bbl. capacity, with Fuller-Kinyon pumps for pumping cement ashore. It is powered with two Atlas-Diesel engines of 350 h. p. at 225 r. p. m., connected to propellers by air operated clutches that slip with an overload. A new hydro-pneumatic control of clutches has been designed by the builders. The propellers, 6 ft. dia., 63-in. pitch, are solid semi-steel. Double cast iron rudders are used foreward and aft of each propeller. A hydro-pneumatic steering gear operates either set independently. There is no wheel, steering being by levers which are synchronized with helm angles. Hull is all steel. Stern is straight sided and flat topped to faciliate the easy entrance and discharge of water to the propellers.—EDMUND SHAW

Seven 120 ft. cement kilns replaced by one 305 ft. in length. *Pit and Quarry*, Dec. 17, 1930, V. 21, No. 6, p. 25-28.—Plant of the Wolverine Portland Cement Co., at Quincy, Mich., in operation since 1899 has been altered numerous times and, with the changes made in 1930, is up to date. Raw material is marl, excavated by a clamshell-bucket dredge and pumped into large wooden barges which are towed from the small chain of lakes, through a barge canal, to the dock at the main plant. Dredging is precluded during about three months each winter. The marl is pumped from barges, by a return-air system, into steel storage tanks of a combined capacity of 540 cu. yd. Samples are taken and clay is added as marl flows to pug-mill. From pug-mill the mixture goes first to a vat and thence to tube-mills. Slurry then flows to another vat and thence to tanks of 90 cu. yd. each. A long screw conveyor takes slurry to the kiln building where it is elevated and fed to kiln which is 305 ft. long by 10 ft. in diam. and is driven by a 100-h. p. motor at one revolution in 58 sec. Gases from kiln are withdrawn by a 10-ft. fan. Coal is dried, pulverized and pumped pneumatically into steel bin whence it is fed to the air stream furnished by a 4-ft. fan, into the firing end of the kiln. Electric recording pyrometer registers the temperature of the exhaust gases. Clinker drops into a pit and thence by two drag-chain conveyors to a pan conveyor which leads to storage. Drag-chains can move the clinker to a cooler which discharges to storage. A monorail hoist with a 1-cu. yd. clamshell bucket reclaims the clinker and discharges it to a scale where gypsum is added. The material is then elevated to feed nine 30-in. mills. Final grinding is in two tube-mills. Cement goes to a 14-ft. air separator and the coarse fraction returns to tube-mills. Finished product is taken by screw conveyors and elevators to a bin storage.—A. J. HOSKIN

Strength properties of cements during 1930. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Some of the problems of coloring concrete. GEORGE RICE. *Concrete Products*, Dec., 1930, V. 39, No. 6, p. 16-17.—This article takes up the coloring of concrete from a scientific standpoint, showing the molecular structure of color used in cement and the manner in which it may be attacked by color-weakening baculli. The author also discusses methods for preventing the weakening of colors, as well as the interaction of one color upon another.—E. S. HANSON

Studies on fundamental synthesis of calcium aluminates and their hydration—Part 7. SHOICHIRO NAGAI AND RYUICHI NAITO. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental Binding, No. 8, p. 315-8B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Heat tests with Jaeneckeit showed that this compound consists at 2642° F. of 1 tricalcium aluminate, 2 dicalcium silicate and 1 CaO. Between 2732° and 2822° F. combination of CaO and dicalcium silicate to tricalcium silicate takes place. Eutectic mixtures of β-$2CaO.SiO_2$ and $5CaO.3Al_2O_3$ were obtained by heating to 2462° F. This mixture has relatively high strength properties. Similar tests were conducted with gehlenit.—A. E. BEITLICH

Hydrates of calcium sulfate. F. KRAUSS AND G. JÖRNS. *Tonind. Ztg.* (Germany), Nov. 24 and 27, 1930, V. 54, No. 94, p. 1467-8, No. 95, p. 1483-4.—The great variability of results reported in literature for hydrates of calcium sulfate occurs because of sensitiveness of these compounds to changes in grain size and to experimental conditions. Previous history is also important. The work of Linck and Jung establishing the zeolitic character of the hemihydrate is confirmed. To prepare pure hemihydrate start with freshly precipitated dihydrate, heat for a short time at its decomposition temperature, cool quickly where two forms are in equilibrium to reduce the production of anhydrite. The transformation took place at 59° C. and at a vapor pressure of 7 mm. in the absence of air.—F. O. ANDEREGG

Action of gypsum, swelling and hydration processes. F. TIPPMANN. *Zement* (Germany), Oct. 30, Nov. 6, 13, 20, 1930, V. 19, No. 44-47, p. 1030-5, 1056-60, 1080-6, 1106-12.—Opinions about effect of gypsum and anhydrite on setting properties of cement differ widely. Berger, Meade and Budnikoff have in general no objection to use of anhydrite as portland cement retarder, while Platzmann and Hart do not favor its use. Comprehensive tests were made to clear up this problem. Four different retarders: (1) raw gypsum rock of high purity, (2) natural pure anhydrite, (3) semihydrate ($CaSO_4.1/2H_2O$), produced from gypsum rock by heating for several hours at 311° F. and (4) stucco, regenerated dihydrate from waste of clay industry. Grinding tests were carried out with a well burned clinker in great laboratory ball mill in lots of about 70 lbs. over 5 hour periods. Additions of retarding agents ranged from 1 to 6 per cent. Obtained products were tested for fineness, setting time, 3-, 7-, 28-day compressive and tensile strength after combination storage, volume constancy, Le Chatelier test and SO_3 content. Additional tests were made with 2.5 per cent raw gypsum rock and 4.5 per cent anhydrite for 16 hours. Temperatures in mill during grinding process were carefully determined. It was found that anhydrite shows less retarding effect on clinker than the other modifications with water of crystallization. Slow setting products were prepared with 2 per cent raw gypsum, 1 per cent semihydrate, 2 per cent stucco and 4 to 5 per cent anhydrite. Constancy of volume was not affected by these high additions. Changes in SO_3 content of samples seem to indicate that a partial thermal dissociation takes place, caused by heat in mill. Regenerated dihydrate is not affected and needs higher temperatures. Author discusses opinions of numerous investigators (Biehl, Platzmann, Killig, Kuhl, Probst, Dorsch, Schmidt and Haehnle) and compares their findings with his own results. He claims that no calcium sulfoaluminate is formed when cement and water react together. This compound was prepared synthetically by treating white portland cement of the Medusa Portland Cement Co. (Ohio), with saturated solution of gypsum. Reactions of calcium sulfoaluminate and semihydrate with water and progress of crystallization with gypsum solutions of different concentrations were studied under microscope. Investigation includes studies of formation of colloidal gels in gypsum solution in connection with swelling tests. Special built apparatus allows microscopic observation of a stirred mixture of cement or cement compounds in gypsum solutions or lime water. Two phases of reaction could be observed: (1) electrolyte causes agglomeration of fine

particles without destroying their structure (apparent swelling process) and (2) true swelling process of solid residue and formation of new substances (silicate needles and aluminate plates) takes place; gypsum has little influence on this phase of reaction, while concentration of calcium hydroxide has great effect. Retarding effect of gypsum seems to be due to fact that a weak gypsum solution retards speed of solution of lime causing slower formation of hydration products. Silicate needles and an abundance of a transparant gelatinous substance which contains silica and lime (probably a pseudo-crystalline or liquid-crystalline modification of a calcium silicate) were found between set cement plates under water storage. Transformation of calcium hydroxide crystals from a colloidal to a crystalline form could be observed under microscope. Synthetic calcium aluminate hydrates were prepared and their optical properties determined. Finally solution phenomena of cement in sugar solutions of different concentrations and formation of crystal phases were examined. Thirty-nine references, 28 microphotographs, 11 graphs and several illustrations are given.—A. E. BEITLICH

PROPERTIES OF CONCRETE

The chemistry of the resistance of concrete to corrosion. WILLI SERKIN. *Betonwerk* (Germany), June 15, 1930, V. 18, No. 24, p. 334.—A discussion of action of soluble portion of cement in connection with insoluble aggregates, and of the influence of water on lime content of cement. Methods now in vogue to protect concrete by integral methods, changing the soluble portions into insoluble, surface and impregnation materials are fully discussed and described. It is cited that as yet no universal method of protecting concrete has been discovered.—H. FRAUENFELDER

Crystal Springs dam in California, 40 years old concrete still watertight. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1022.—Data of inspection of Crystal Springs dam, which is still in perfect condition after a 40 year period, indicate that this condition is due to an extraordinarily careful construction control, use of perfect aggregates and systematic treatment of finished concrete.—A. E. BEITLICH

Overheating of aggregates found detrimental to concrete. WILLIAM H. BACHELDER. *Eng. News Record*, Dec. 18, 1930, V. 105, No. 25, p. 973.—In cold weather it is often necessary to heat concrete aggregates. Some specifications require that concrete when placed shall have temperature of from 70 to 140° F. This temperature has some effect on chemical reaction that takes place when cement sets. It has been assumed that effect of higher temperatures on resulting concrete is beneficial. A series of tests in Minnesota highway department laboratory showed that temperatures above 70° F. should not be used if the full development of strength inherent to concrete is to be obtained. Concrete placed at 130° F. loses approximately 20 per cent of its strength.—D. E. LARSON

Artificial influencing of strength of concrete—Part 3. C. R. PLATZMANN. *Zement* (Germany), Nov. 27, 1930, V. 19, No. 48, p. 1139-41.—Several surface hardeners for concrete are discussed. Special consideration is given to "Metalicron," a product made by Master Builders in Cleveland, Ohio. Properties of this material were carefully studied and abrasion of concrete specimens treated in different manners were measured. Photographs show corrosive influence of lactic acid, sugar solutions and acids from fats and oils. Microphotograph of Metalicron is given. A surprisingly high increase in strength was found of specimens stored in a solution containing 8 per cent sodium sulphate and 2 per cent magnesium sulphate.—A. E. BEITLICH

Retempered mortar. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 696.—Results are given of tests made at the Building Research Station to determine effect on strength of mortar of remixing cement mortars up to periods of 30 to 45 minutes after mixing and after initial set has taken place. The rate of hardening of cement and consistency of mix are important factors in deciding final result. For slow hardening portland cement it may be possible to regauge cement mortar for periods up to possibly 4 hr. without loss in strength, while for rapid-hardening and aluminous cements there is risk of loss in strength occurring due to retempering. Two series of tests were carried out at Building Research Station. One series for normal slow hardening portland cement and the other for rapid hardening portland cement. Values are given of strengths for varying times of remixing. The general conclusion made was that the process of retempering cement mortar and concrete cannot be recommended.—JOSEPH MARIN

Control of concrete construction. "Die Baukontrolle in Betonbau." BREBERA. 1929, Eduard Gregrasin, Prague (Czecho-Slovak Republic). Reviewed in *Bautechnik* (Germany), Sept. 19, 1930, V. 8, No. 41, p. 636.—Tentative directions for production of concrete are derived under consideration of all experiences in recent years with big concrete construction jobs. After emphasizing necessity of construction control, author discusses factors which influence quality and strength of concrete. Ball-test by Heinzel for determination of setting time is described and also a method for testing binding properties of cement, found by determining flexural strength of 10-in. long mortar prisms. Special consideration is given to chapters on aggregates, methods for determination of impurities, practical gradations and proportioning according to Abrams fineness modulus. Mixing water and influence of natural water content of aggregates on strength are considered. Final chapter discusses equipment and apparatus for perfect concrete mixing.—A. E. BEITLICH

Mixing proportions not only factor affecting concrete strength. ALBERT MERCIOT. *Constructeur de Ciment Armé*, Oct., 1930, No. 133, p. 217-219.—Somewhat out-of-date to American readers are arguments collected here in favor of field control of concrete. Concrete strength should not be conditioned by mixing proportions, relative proportions of sand and gravel, quantity of mixing water, or method of working concrete. It should be sufficient to simply specify that after a certain lapse of time concrete on the job should show a resistance to rupture, which, in accordance with factor of safety desired, corresponds to working stresses assumed in the design. This will give great freedom to contractor in choice of his materials and methods. Uniformity of concrete will be secured on the job. Scientific control on the job will eliminate complaints of fraud. Already there exist in France companies which will give this type of service on reinforced concrete construction work.—M. A. CORBIN

Strength properties of cements during 1930. ALEXANDER HASCH AND LEOPOLD RISTER. *Osterr. Bauzeitung* (Austria), Nov. 15, 1930, V. 6, No. 46, p. 752.—Compressive and tensile strength data of cements tested during 1930 by Austrian Institute for Testing Materials in Vienna are tabulated. Such cements are ordinary portland cement, high early strength portland cement, Kuhl cement and several foreign brands of fused high alumina cements. Additional durability tests were carried out by storing specimens in various aggressive solutions—sulphuric acid (5 per cent), magnesium sulphate (5 per cent), ammonium nitrate (5 per cent), sodium nitrate (1 per cent) and saturated gypsum solution. Solutions were renewed every 2 weeks.—A. E. BEITLICH

Effect of shrinkage in large concrete dams. M. A. RENAUD. *Annales Ponts Chaussees* (France), March-April, 1930, V. 1, No. 2, p. 144-179.—Fissures produced by shrinkage of concrete in dams are important in their extent. Study of how fissures were produced can only be made after failure has taken

place, and then there is doubt as to what actually took place in the interior. Engineers in charge of the Saint-Marc dam observed concrete while dam was in different stages of completion and were thus able to note any fissures that formed and to remove them in time. Dam is of gravity type, 145 ft. high, and contains 97,500 cu. yd. of masonry. Method of construction is explained and stages in placing of concrete indicated by diagrams. Figures illustrate age of concrete of different parts of dam and proportions of aggregates. Extent of fissures was determined by means of colored water and detailed account is given of formation, growth and location of fissures. Extent of fissures is shown by means of diagrams of the cross-section, for different periods of time after the first evidence of a crack was noticed. Main deduction made from observations was that fissures were due to shrinkage of concrete, for construction was not advanced sufficiently for dead load of masonry to produce cracks, nor was temperature sufficiently low to cause fissures. Proposed measures to protect against formation of new fissures, and method of repairing dam in which fissures are already formed are outlined. Observations taken on Saint-Marc dam point out necessity of observing dam during its construction for fissures that might form. Fissures in another dam that has been in service for two years were found analogous to those in Saint-Marc dam.—JOSEPH MARIN

Valuation of shrinkage numbers of light-concrete. A. HUMMEL. *Zement* (Germany), Nov. 6, 1930, V. 19, No. 45, p. 1062-6.—Shrinkage of mortar and concrete is due to (1) shrinkage of cement, (2) shrinkage of aggregates, (3) gradation of aggregates and (4) amount of gauging water. Air moisture, temperature conditions and structure of concrete also affect shrinkage. Shrinkage can be reduced by use of lean mixtures with little gauging water; shrinkage process can be retarded by keeping concrete wet until a considerable hardness has been produced. For light-weight concrete, aggregates must have small coefficient of contraction besides heat insulating properties and light weight per cu. ft. Frequently shrinkage numbers of light concrete can be found which are much higher than those of normal concrete. A formula for comparison of shrinkage numbers is derived based on tensile strength, specific shrinkage and modulus of elasticity. Several different concretes are investigated from this point of view.—A. E. BEITLICH

Glacial sand-gravel aggregate found suitable for paving work. (See MATERIALS—AGGREGATES.)

Action of gypsum, swelling and hydration process. (See MATERIALS—MISCELLANEOUS.)

Advantage and versatility of cast stone. (See ARCHITECTURAL DESIGN.)

ENGINEERING DESIGN

BRIDGES

A method of arch design. G. P. MANNING. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 687-689.—Paper deals with "combined moment envelopes," diagrams representing total positive and negative moments for points along the arch. Moments plotted include those due to dead load, live load, crown drop, and temperature. The diagram given is the "envelope" for a specific case. The arch treated is as follows: span, 150 ft., rise 30 ft., open spandrel type with floor and roadway weighing 250 lb. per sq. ft., ribs spaced at 10 ft. centers, live load consists of one standard train including 50 per cent impact, E_c = 2,000,000, I_c = 545,000 in.^{4}, cross section of the rib at the crown 30 in. wide by 48 in. deep with 1.5 per cent steel top and bottom, and a temperature range of + or — 30 degrees. The moments were calculated for two types of profiles and comparison made.—JOSEPH MARIN

BUILDINGS

Large building structure with shock absorption. *Osterr. Bauzeitung* (Austria), Nov. 22, 1930, V. 6, No. 47, p. 767.—New construction principle was used for construction of "Rhenania-Ossag"-house in Berlin (Germany). No direct connection exists between inner structure and outer walls which fact makes this office building shock proof and reduces considerably, vibrations caused by street traffic.—A. E. BEITLICH

Reinforced concrete skeleton structure for an apartment and office building. JOS. HOLZAPFEL. *Zement* (Germany), Nov. 20, 1930, V. 19, No. 47, p. 1114-7.—Design and construction of building at street corner in Munich (Germany) was extremely difficult on account of very unfavorable underground and narrow site of construction. The building is 93.5 ft. long but only 16.4 ft. wide. Construction as reinforced concrete skeleton structure was only possible solution. Details are given of dimensions and reinforcements of structural parts.—A. E. BEITLICH

Cleveland railroads dedicate Union Terminal. Foundations for the Cleveland Union Terminal. C. P. MARSH. *Civil Engineering*, Nov., 1930, V. 1, No. 2, p. 77-88.—This recently completed $88,000,000 project included a 52-story tower supported on foundations extending 204 ft. below rail level. Quantities included 500,000 cu. yd. of concrete and 100,000 tons of steel work. The terminal serves the New York Central, Big Four, and Nickel Plate railroads. Preliminary foundation investigations involved much work of a pioneer nature including library studies, 8,000 linear ft. of test holes, soil analyses, and bearing tests on soil and rock samples and field tests of soil in place. This soil was fine sand containing about 13 per cent clay and 3 tons per sq. ft. was the final pressure allowed for retaining walls and spread footings placed upon it. At 60 or 70 ft. below basements there is hardpan of compact clay mixed with gravel on which a pressure or 7½ tons per sq. ft. was attained. The tower is supported on 16 concrete piers carried to rock for which 100 tons per sq. ft. is allowed. About 1,000 columns are supported on spread footings, the largest of which is 32 by 41 ft. and carries a load of 6,300,000 lb. Tower foundations carry loads from 6,300,000 to 8,900,000 lb. and vary in diameter from 8 ft. 8 in. to 10 ft. 4 in. Minimum pier diameter is 4 ft. and is enlarged only as may be required by limiting stress of 400 lb. per sq. in. in the 1:2:4 concrete (below the spiral) is exceeded. Piers resting on clay have bell bottoms. Foundations cost about $1.25 per cu. ft. and quantities were about 74,000 cu. yd. of concrete and 3,100 tons of reinforcing steel. The paper on foundations outlines the problems and mentions factors such as bearing power, skin friction, subsidence, and cost which entered into their solution.—H. J. GILKEY

Examples of mushroom floors in France. W. CHERRE. *Constructeur de Ciment Armé*, Nov., 1930, No. 134, p. 245-247.—Strasbourg firm "Zublin" has erected numerous mushroom floors in Alsace, including building of the Société Alsacienne de Constructions Méchaniques at Mulhouse. It measures 280 by 82 ft. and comprises 7 mushroom floors. Distance between axes of columns is 23 ft. in longitudinal direction and 26.25 ft. in direction of width. Rectangular slabs measure 23 by 26 ft. and represent considerable spans for reinforced concrete construction, in view of the heavy loading. Loading on ground floor is 1635 lb. per sq. ft. and that of floor above is 410 lb. per sq. ft. Depth of slabs is relatively low, being 20 in. for the ground floor; 10 in. for first floor and 9 in. for floor above. The computations were made in research department of Zublin Co., according to method of Prof. Ritter of the Zurich Polytechnic School. The building was completed in 1926. Another example, built in 1913 by the same company, is warehouse of Société d'expeditions Léon Weiss at Strasbourg. The surface area of the mushroom floors is 86,000

sq. ft.; they are designed to support loads varying from 305 to 450 lb. per sq. ft. The span is 15 ft., the depth of slab varies from 8½ to 9¾ in. Another example is the covering of a reservoir of Dollfus-Mieg Co. at Belfort, built in 1927. The columns are spaced 11.5 ft. in one direction and 11.1 ft. in the other. Depth of slab is about 6 in., designed to withstand a loading of 490 lb. per sq. ft. The author urges further study of this type of construction.—M. A. CORBIN

Continuous slabs reinforced in two directions. W. S. GRAY. *Concrete Constr. Eng.* (England), Dec., 1930, v. 25, No. 12, p. 661-669.—It is often assumed in designing interior panels of uniformly loaded continuous slabs reinforced in two directions that the positive moment is 0.8 times that calculated for a freely-supported panel. There is little justification for this assumption, its derivation being from analogy with beams or slabs reinforced in one direction only. Experience has proved this method is on safe side, but gives no guide to factor of safety. It is well known that method of design of slabs reinforced in two directions by dividing slab into two series of rectangular strips is inaccurate. A more economical design is obtained if reinforcement is designed to resist torsional moments at all points in the slab. Formulae were developed by Dr. H. Marcus which are applicable to mesh-reinforced rectangular slabs with any number of fixed edges and may be adapted to deal with slabs continuous in one or more directions. This method of calculating moments is specified now in German regulations for reinforced concrete. Method of obtaining values of moments is: Slab is assumed divided into two bands of rectangular strips considered as acting independently, and expressions for moments obtained. Assuming equal values of *EI* for the two strips at right angles by equating values for deflections for each strip values of moments can be obtained for each strip in terms of uniform load and length of two strips. These expressions for moments are multiplied by reduction coefficients if corners are restrained, thus allowing for torsional moments and restrained deflection of strip. Curves are plotted showing relation between values of reduction factors and ratios of sides of slab. Ratios of sides of slab of 0.5 to 2.0 were considered. Curves are plotted for different degrees of restraint. Following cases are treated: slab freely supported, fixed on one side, fixed on two opposite sides, fixed on two adjacent sides, fixed on three sides and fixed on four sides. Method is given of obtaining the moments caused by dead load and live load in continuous slabs extending over several panels. Method used is similar to that of obtaining influence lines for continuous frames with unsymmetrical loading, and considers dead load plus live load in alternate panels. Expressions for moments in interior panels can be obtained using the above curves for interior slabs under loading considered fixed on two sides and simply supported on other two. Values of moments due to live load on alternate panels can be similarly obtained from curves for case of panels simply supported. Combining two sets of values given by two loadings moments due to dead plus live load can be obtained. An example explains method of computation for a six-panel continuous frame. Method has advantage over other methods that it is simple. In other solutions of problem effect of continuity is arbitrarily assumed or treatment is too complicated for ordinary office design. The above paper deals only with point load concentrations. —JOSEPH MARIN

DAMS

Hoover dam. ELWOOD MEAD. *Civil Engineering*, Oct., 1930, V. 1, No. 1, p. 3-8.—The paper outlines the Boulder Canyon Project in its larger and in its preliminary aspects. Nevertheless all aspects of a project that includes a 700 ft. concrete dam, a reservoir eight times larger than the Assuan, machinery to develop a million and a quarter horse power, an All-American Canal to irrigate nearly a million acres with an excavation one fourth that of Panama

Canal all at total cost of $165,000,000, should be of interest to readers of the Journal and it is deemed proper that their attention should be called to this treatment which in itself has little to say about many vital concrete features of the project.—H. J. GILKEY

Hydro power station of Rocky River in Connecticut. *Die Bautechnik* (Germany), Aug. 1, 1930, V. 8, No. 33, p. 510-2.—Hydro power development of Rocky River, built by Connecticut Light and Power Co., consists of a water reservoir covering about 8.49 sq. miles. Capacity of lake is 218.5 millions cu. yd. Earth dam rests on concrete wall founded 5 ft. deep on sound rocks. A canal 3400 ft. long on east side of river leads to intake and power station. A generator, producing 30,000 KW. and 2 centrifugal pumps are installed. Plant was designed for a later addition of a second unit.—A. E. BEITLICH

The "cracked" cantilever in dam design. B. F. JAKOBSEN. *Eng. Con.*, Dec., 1930, p. 439-441.—In a brief historical review of the lack of agreement between calculated and actual deflections of dams, the author discusses overlooked factors which tend to materially increase calculated deflections. Shear deformation, yielding of foundation, effect of swelling of upstream side, and time yield of concrete may all combine to produce deformation ten times greater than that found when only bending, calculated from initial modulus of elasticity, is included. This agrees with present-day theory. References given include L. Jorgensen, Frederik Vogt, W. A. Slater, F. R. McMillan, Aarskog, A. H. Fuller and C. C. More. "Cracked" cantilever and the horizontal "hinge" are treated in detail. Mathematical analyses show that ability of "hinge" to produce more favorable stress distribution is quite negligible. Water entering a crack tends to produce tension at bottom of crack whereas compression might have existed had the crack not been present. Shear is greater due to a crack. Calculations of deflections of dams cannot be made with any degree of precision because the great increase in deformation due to plastic yielding is dependent upon the load history of the dam. Temperature changes and water load on the reservoir may affect the deflection of the dam.—N. H. ROY

Water power stations of river Liri in middle Italy. W. VIESER. *Bautechnik* (Germany), Sept. 12, 1930, V. 8, No. 39, p. 580-3.—Favorable water and geological conditions led to construction of several hydro power stations in course of river Liri between Isola del Liri and S. Giovanni Incarico (Italy). Amount of water is practically constant during almost 6 months. Catching area for upper stations is 586.7 sq. miles, for lower stations 1274 sq. miles. Four stations were built in upper part of river. Structures of power plants in middle and lower part of river Liri include gravity dam in Portefiume which forms a water reservoir of 0.58 sq. miles. Dam is 103.4 ft. high, 262.5 ft. long and has volume of 26,160 cu. yds. Downstream side of dam is protected against force of water by sinking of 27 reinforced concrete caissons, each 36 ft. long and 10.3 ft. wide, in 3 rows. Water canal has cross-section of 35.9 sq. yd. and is capable of delivering 109.9 cu. yd. per sec. Power house is 121 by 33 ft. Hydro power station in Ceprano is fed by two water reservoirs connected by canal. Dam in valley of Liri is of gravity type. 283.8 ft. long, 106.6 ft. high and has 103.4 ft. spillway. Reinforced concrete dam in Rio Cancello is 132.8 ft. long and 72.2 ft. high. It has 3 flood gates, each 6.5 ft. square. Connecting canal is lined with reinforced concrete. Water is delivered to power station through cylindrical reinforced concrete pressure pipe 12.5 ft. diameter. Reinforced concrete power house is 196.8 ft. long. Additional information is furnished of construction of stations. Several small dams, weirs and pressure pipe lines in reinforced concrete were built in Fontana Piri, Fontecupa, Serelle and Liri-Fibreno (Italy).—A. E. BEITLICH

MISCELLANEOUS

Calculation practice for reinforced concrete. "Pratique du Calcul

du Beton Armé." G. MAGNEL. 1930, Van Rysselberghe and Rombaut, Paris (France), 3rd part, 360 pp., 12 tables, Frs. 140. Reviewed in *Bautechnik* (Germany), Sept. 19, 1930, V. 8, No. 41, p. 635.—Main subject of book is theory and calculation of concrete and reinforced concrete arches with 2 and 3 hinges. Investigation is carried out for vertical and horizontal loads.—A. E. BEITLICH

Research advances in civil engineering. ALFRED D. FLINN. *Civil Engineering*, Oct., 1930, V. 1, No. 1, p. 14-16.—Attention is called to the value of research as a means for settling differences of opinion and eliminating waste. The varied agencies engaging in research and a great many specific research problems are mentioned. Among those of greatest probable interest which have been sponsored by the Engineering Foundation in recent years are: research on arch dams and dams in general, concrete arches, earth foundations, earthquake stresses, retaining walls, cements, flow of concrete, testing small models, etc. It is pointed out shat all research should begin in the library.—H. J. GILKEY

Reinforced concrete transverse ties for railroads. ROUDOLF. *Bautechnik* (Germany), Aug. 1, 1930, V. 8, No. 33, p. 507-8.—Cracks occur usually in middle of reinforced concrete ties, made of one single piece, no matter how wide they are. Twin ties show greatest damage. Concrete ties are in general made shorter than wooden ties to decrease their weight. New type of ties, system Roudolf, consists of two parts, 8.8 ft. long, connected by center hinge which prevents cracks. Another advantage is great resistance agzinst horizontal stresses. Railroad ties should consist of not more than two parts.—A. E. BEITLICH

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete*, Dec., 1930, V. 37, No. 6, p. 40-41.—Chart O_1 may be used to design tied columns according to the A. C. I. Code. Chart M_1 may be used in designing spirally reinforced columns according to the Joint Committee or the A. C. I. Code. These two charts complete the series. There are now available design charts for beams, slabs, and columns for 2000, 2500, and 3000 lb. concrete.—N. H. ROY

The proposed underground railway system for City of Rome (Italy) and its reinforced concrete structures. JOSEF STELLINGWERFF. *Osterr. Bauzeitung* (Austria), Nov. 22, 1930, V. 6, No. 47, p. 763.—Extensive underground system is projected and designed for Rome, street traffic of which has increased more than five times during last 10 years. Three main lines are 6.73, 3.31 and 4.91 miles long. For tubes and superstructures reinforced concrete will be used, made of high early strength cement. All structures will be proof against aircraft attacks. (cf. *L'industria Italiana del Cemento* (Italy), May, 1930.)—A. E. BEITLICH

Concrete aqueduct irrigates half million acres. GEORGE P. FRENCH. *Concrete*, Dec., 1930, V. 37, No. 6, p. 39.—A reinforced concrete aqueduct across a dried-up river bottom has been built in the neighborhood of Brooks, Alberta, by the Canadian Pacific Railway. The structure consists of two miles of concrete flume, in basin form, 20 ft. wide by 10 ft. deep, supported by steel and concrete trestles. The maximum height of structure is 50 ft. A large inverted siphon of reinforced concrete carries the water under the railway tracks.—N. H. ROY

Elementary calculation of continuous beams. N. RAUBAL. *Osterr. Bauzeitung* (Austria), Nov. 8, 1930, V. 6, No. 45, p. 735-7.—Formulas are derived and examples calculated which illustrate amount and direction of stresses in different kinds of continuous beams under various loads.—A. E. BEITLICH

Structural design of Chicago's subway. *Eng. News Record*, Dec. 11, 1930, V. 105, No. 24, p. 932-933.—Plans for Chicago's subway have been completed to point of showing how structure is to be built. Essentially, project consists of a four-track subway $3^1/_3$ miles long under State St., extending from Eighteenth St. north across the Chicago River to Chicago Ave. and turning west along Chicago Ave. to Franklin St. In general, the subway consists of a roof and walls of concrete jack arches supported by a structural steel frame and floor of reinforced concrete. The latter is reinforced against an upward hydraulic pressure of 90 lb. per sq. ft. The steel bents are spaced 5½ ft. on centers, and in the station areas they are placed at every third bent or 16½ ft. on centers. The river tunnels 1900 ft. long, consist of one double-track and two single-track tubes. The subaqueous section of the tunnels will be steel tubes sunk in place and lined with reinforced concrete, similar to those in New York under the Harlem River and the LaSalle St. tunnel in Chicago.—D. E. Larson

Roads and Pavements

New methods of measuring concussions caused by traffic on roads. Risch. *Betonstrasse* (Germany), Aug., 1930, V. 5, No. 8, p. 194-198.—In general meeting of "Studiengeselishaft fur Automobilstrassenbau" (Institute for the study of the building of automobile streets) the author cited that with increased use of heavy trucks, complaints of damage to buildings and destruction of pipe-lines below the street, claimed to be caused by traffic concussions, have caused desire to measure these concussions to determine truth of these complaints. Instruments used in the past, mostly built according to the principle of the seismometer were not suited. The curves drawn by these instruments were more or less deformed through the vibration of the mass of the instruments. In the "Ambronnschen Beschleunigungs messer" (acceleration meter) is used the Piezoelectricity of closely piled up silicate crystals. The discharge is artificially increased and measured with a galvanometer, actions of which are filmed and the film timed. This instrument is also very easily handled. Several measuring bodies can be placed in different places and directions and the results taken on one film. General principles which have to be taken into consideration in judging results include condition of soil and its ability to conduct concussions.—R. J. Dietrich

Sewers

Concrete sewer design adopted as standard after severe test. *Eng. News Record*, Dec. 11, 1930, V. 105, No. 24, p. 937-938.—The city of Richmond, Va., is to construct about 25,000-lin. ft. of large sewer within the next five years. Studies have led to the development of a horseshoe-type reinforced-concrete section equivalent in capacity to circular pipe with diameters from 60 to 144 in. Salient features of the design are: (1) Vertical and horizontal diameters are practically equal to diameter of an equivalent circular section. (2) Invert acts as no part of structural design except where foundations are soft; thus the washing out of invert does not affect stability of arch and walls. (3) Sheathing of trench can be used as the outside form up to springing line. (4) The design is sufficiently flexible to be adaptable to soils of different bearing capacities. As a final check on design, an 8-ft. section equivalent in size to an 84-in. circle was tested by piling pig iron on a platform resting on the upper quadrant of the arch. Hair cracks developed in center of intrados at both ends when load reached 129,419 lb. The ultimate load was about 153,460 lb.—D. E. Larson

ARCHITECTURAL DESIGN

Reinforced concrete hall designed for clerestory lighting. *Concrete*, Dec., 1930, V. 37, No. 6, p. 34.—Design of Horticultural hall departs somewhat from London building practice in that four tiers of vertical windows rise from the roof arches in step-fashion. Daylight is thus admitted into the large hall by a complete system of clerestory lights. There are several examples of this system on the continent. The building is of reinforced concrete. The foundation rests on clean dry sand.—N. H. ROY

Advantages and versatility of cast stone. WALTER OBST. *Betonwerk* (Germany), Aug. 30, 1930, V. 18, No. 35, p. 487.—Real or imitation, natural or artificial, original or substitute, these are the contrasts encountered in present day enterprises. Many synthetic products today have qualities all their own and do not in the main, imitate the natural product, but in many cases excel. Modern cast stone is such product. It is now used in first-class undertakings, in sculpture and monuments. Cast stone has a structure and texture all its own. It has attained a solid position in the public mind. The outstanding qualities of cast stone are texture, density, color, durability and ease of ornamentation.—H. FRAUENFELDER

Reinforced concrete and aesthetics. E. BLUNCK. *Report of the 33rd main meeting of German Concrete Association*, March 17, 1930, p. 219-35.—Fundamental difference between reinforced concrete and other structural materials is fact that these structures have monolithic character; supporting and supported parts, horizontal and vertical parts are rigidly connected. This construction principle allows greater freedom of shaping. Correct valuation of modern architectural forms is rendered more difficult by traditional opinions, which compare structures with old standards. Uniformity, contrasts, symmetry and rhythm in form and dimensions are most important factors which govern modern aesthetic reinforced concrete structures. Author shows in 15 illustrations selection of recently built structures which are outstanding in their architectural design, and criticises their forms. Such structures are industrial and office buildings, water towers, arch bridges, churches, locks and dwelling houses. Great is number of mistakes and unaesthetic designs, and much fundamental explaining work among contractors has to be done in future.—A. E. BEITLICH

Concrete and reinforced concrete for construction of dwelling houses. MUELLER. *Report of the 33rd main meeting of German Concrete Association*, March 17, 1930, p. 192-219.—Use of concrete for construction of dwelling houses is not very common in Germany on account of its high price and less heat insulating properties than brick walls. Author discusses efforts to overcome these difficulties and describes number of structures recently erected. Concrete with pumice stone as aggregates and air-concrete (large voids) were used for small apartment buildings. More favored is construction with small or larger concrete form bricks, which method eliminates high form costs. Of advantage is shorter time period for drying out of walls. Author describes construction method which uses concrete flat slabs which are especially suited for ceilings forming heat insulating air cells. Reinforced concrete is frequently used for modern apartment house construction allowing greater varieties in architectonical forms. More economic form methods and improved equipment must be developed for further progress.—A. E. BEITLICH

FIELD CONSTRUCTION

BRIDGES

Reinforced concrete bridges of the German railroad concern (Reichsbahn). GEORG MUENZ. *Bautechnik* (Germany), July 11, 1930, V. 8, No. 30,

p. 463-4.—Construction of 3 reinforced concrete arch bridges with spans of 42.6, 49.2 and 82.0 ft. are described and details of their reinforcements are given. Iron portland cement, which gave compressive strength of 7600 lbs. per sq. in. in 28 days, was used in a 1:2½:1½ mix with gravel and broken granite of ¼ to ¾ in. size. Mixing water was kept as low as possible, and concrete was continuously tested during entire construction period.—A. E. BEITLICH

New construction of a viaduct of the Hannover Station in Hamburg (Germany). BLUNCK. *Bautechnik* (Germany), Sept. 5, 26, 1930, V. 8, No. 38-42, p. 563-5, 639-41.—Main railroad line crosses freight tracks in semi-circle with radius of 984.3 ft. Old viaduct structure was 393.7 ft. long and rested on masonry piers. Smoke from engines had done considerable damage to railroad superstructure and piers which fact demanded entire reconstruction of viaduct. Solid roadbed is now constructed, on reinforced concrete two-hinged frame construction, made of 1:4 mix. Most all of old piers were provided with reinforced concrete casing. Road slab consists of transverse I-beams, placed 25.5 in. apart, totally encased in reinforced concrete (1:5). Rods (0.7 in. in diam.) are placed above and below beams spaced 12 in. o. c. Protecting layer of 1-in. neat cement and two layers of asphalt and bitumen insulation were placed. Broken rocks of railroad bed are filled on top and between two reinforced concrete embankments which contain pipes for power and telegraph cables. Drafts for engine smoke and drain pipes were built in road slab. Foundations for frame supports were made of 1:6 mix. Conditions were very unfavorable for storage and preparation of structural materials on account of lack of space. Concrete mixer was located on elevated platform. Cement and aggregates were delivered to mixer from small storage bins by elevator. Small traveling cable crane transported concrete from mixer to workmen. Aggregates consisted of 2.5 parts of gravel and 0.5 parts of broken basalt (0.6 to 1 in.). Entire construction program was carried out without interruption of heavy passenger and freight traffic (over 180 trains per day).—A. E. BEITLICH

Southern Pacific Company's Suisun Bay bridge. W. H. KIRKBRIDE. *Proc. Am. Soc. C. E.*, Aug., 1930, V. 56, No. 6, p. 1307-1318.—Double track railroad bridge replaces two train ferrys on main line of Southern Pacific across Carquinez Straits between Benicia and Port Costa. There is possibility that inactive fault plane crosses site and construction was planned to resist earthquake shock by having centers of gravity of piers as low as possible. Specified 28-day concrete strengths were 2500 and 400 lb. per sq. in. in compression and tension respectively. Steel reinforcement at 30 lb. per cu. yd. of concrete was placed to resist accelerations of 5 ft. per sec. which is twice that of the 1906 San Francisco quake and 1½ times the Japanese quake of 1923. Steel superstructures were planned for similar intensities and were notched into heavy monolithic piers. Swift current, deep water and soft mud bay-bottom called for unique foundation methods. From preliminary octagonal pile platform, 81-ft. diameter steel shells were lowered through water into mud from 15 to 30 ft. Shells were constructed of bolted 10 ft. sections and were extended above water level after which they were filled with sand dredged from river. Caissons were then sunk through sand, inside of shell and properly grounded on rock. A 2-yd. floating concrete plant with 104-ft. tower constituted nucleus of concreting equipment.—H. J. GILKEY

BUILDINGS

Farm storage houses—a market for concrete products—ROBERT H. MOORE. *Concrete Products*, Dec., 1930, V. 39, No. 6, p. 14-15.—This article details the construction of two concrete tile farm storage houses, one with cinders and one with Haydite as aggregate. In first one constructed the original

idea was to build it with double walls of 8 by 8 by 16-in. block with a dead air space between. It was found that a wall of this kind would cost approximately 38¢ per sq. ft. The block manufacturer, however, found that a triple wall could be built of 4 by 8 by 16-in. partition tile for about 36¢ per sq. ft. This would not only give better wall with two air spaces instead of one but would also reduce total wall thickness from 18 in. to 16 in. This method of construction was followed on both buildings.—E. S. HANSON

Development of reinforced concrete construction in Finland and introduction of construction control. OTTO WEYERSTALL. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 112-37.—One of first great reinforced concrete structures was saw mill in Lauritsala, a two-story building, 185 ft. long and 100 ft. wide with slab floor construction. Span of beams is about 50 ft. Development was very rapidly until 1911, when first construction control was established. Conditions in Finland are very favorable for reinforced concrete construction on account of its great natural sources for sand and clean aggregates. Of disadvantage are climatic conditions, since cold weather periods predominate. Several modern cement plants furnish excellent cements with high early strength properties. Specifications for testing cement and concrete were issued a few years ago and follow in general lines of German specifications. Great number of structures are described and illustrated showing progress made in recent years.—A. E. BEITLICH

Speedy erection of structural frame proves value of inspection. B. L. MEHANDRU. *Concrete*, Jan., 1931, V. 38, No. 1, p. 21-23.—Erection of Indiana hotel structure demonstrated that co-operative effort between architect, owner and contractor benefits all. Commercial inspection engineers were retained. Building is 12 stories high, reinforced concrete structural frame and floors, 83 by 69 ft. in plan, requiring 3,500 cu. yd. of concrete. Concrete mixing plant including ½-yd. mixer was at rear of building. Concrete was mixed in 2-sack batches yielding average of 11.3 cu. yd. Tubular chute distributed concrete direct from light steel frame hoisting tower. Architect's specifications called for 2,000-lb. concrete in floors, 3,000 lbs. for columns in first five stories and 2,500 lbs. above fifth floor. Both floor and column concrete above fifth floor was designed for 28-day strength of 2,500 lb. per sq. in. Proportions selected yielded 5.65 cu. ft. in place, per sack. Water-cement ratio was 7¾ gal. to the sack. Slump averaged 7½ in. Complete field testing equipment, including molds, was used on job. Cylinders intended for 7-day test were cured at job 4 days, then shipped to laboratory and tested. Twenty-eight day cylinders cured same length of time at job, then shipped to laboratory and cured in moist room. Average strengths were 1,790 and 2,785 lbs. respectively. Although concreting operations were started 9 days behind schedule, roof slab was concreted July 18, 9 days ahead of schedule. Concreting work made ordinary progress in first and second floor construction, second floor having been concreted June 5th. Third floor concrete placed on June 11. Thereafter a floor was concreted every Tuesday and Friday, until the twelfth floor was placed July 12. Run from third to twelfth floor equivalent to story every 2½ days. Actual schedule maintained from third to twelfth floors, inclusive, left 1½ days for moving and framing form work. Time required for concreting floor, 8 hours. Period from Monday morning to Tuesday noon was occupied in getting ready for Tuesday concreting (afternoon and evening). Friday concreting preparations were made on Wednesday and Thursday.—C. BACHMANN

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Dec., 1930, V. 37, No. 6, p. 35-37.—Considering difficulties of establishing necessary bases on which cost of form work may be estimated in advance, best method is to assume certain average conditions and express required form work for given area in terms of material quantities and hours of labor. Reader may then follow similar methods in figuring his form work problems and arrive at es-

timated cost by applying prevailing unit prices. Greatest variation in cost arises from number of times lumber is used. Comparing 2 and 6-story reinforced concrete building of same ground area, in both cases, floor form panels, beam sides and column forms will be made in sufficient quantity to concrete two floors at a time. On 2-story building forms are used once; on 6-story three times. Cost of material for floor form panels, beam sides, and column forms is as much for 2-story as for 6-story building. Experienced designer makes column sizes in 2-story building alike in both stories, reducing reinforcement in upper columns, and dimensions of roof beams and girders (part below the bottom of the slab) same as floor beams and girders, requiring no patching or altering of form work. Similarly in 6-story building, designer will maintain same column sizes from top to bottom and same beam and girder dimensions below bottom of floor and roof slabs. Following points are guide for estimator: (1) For foundation walls and wall footings of building of moderate ground area, provide sufficient form material for entire area. In building having large ground area, where foundation form work can be erected to keep mixing plant going for about three days, put up only part of form work, moving it second day after concreting is completed. (2) For column footings, make up only enough footing forms so that they can be used four or five times. (3) Forms for columns, for sides of beams and girders, for ordinary floor slabs, for flat-slab floor construction, and walls above grade, should be made up in sufficient quantity for two stories if schedule contemplates one story per week. If two stories per week, make up sufficient forms for three stories. (4) Material for posts, jack frames, braces and other material required to support floor form work must be supplied in sufficient quantity for three stories if schedule calls for one story a week; sufficient for four stories if schedule calls for two stories per week. (5) For beam and girder bottoms which are left in place, along with reshoring timbers and braces, much longer than remaining form work. Quantity of material must be sufficient for four stories if schedule calls for one story per week, and for seven stories if schedule calls for two stories per week. (6) Material for rough structural parts of stairways need not be ordered separately, since rough form lumber can be salvaged from material already used.—C. BACHMANN

DAMS

Salt Springs dam. O. W. PETERSON. *Proc. Am. Soc. C. E.*, Aug., 1930, V. 56, No. 6, p. 1319-1334.—Concrete faced, 328 ft. (above foundation) rock fill dam of Pacific Gas and Electric Co., 70 miles east of Sacramento, Cal., is to impound 130,000 acre ft. to develop all economic available power on Mokelumne River. The main concrete feature is flexible reinforced concrete upper facing 1 ft. thick at top and 3 ft. thick at bottom where it joins the concrete cutoff wall. The average slope of upstream face is 1.3 to 1, top width 15 ft., and bottom 900 ft. along axis of stream bed. Crest length is 900 ft. at elevation of 3959. Of 2,900,000 cu. yd. of rock, 220,000 (7.5 per cent) is derrick placed, 15 ft. thick, upon which the concrete facing is laid. The facing is divided into independent 60-ft. squares. Edges rest on concrete supports poured in grooves left in the rock backing. Two layers of 1-in. round bars at 9 in. o. c. each way reinforce lowest layer and this is gradually reduced to one layer of 1-in. sq. bars 1 ft. o. c. at top. Joints between panels consist of soft copper seals embedded in concrete. Concrete 3,000 lb. at 28 days was designed for maximum density with 5 bags cement per cu. yd. Dam is slightly arched in plan to decrease tension in the concrete facing. Cutoff wall is 6 ft. thick and extends from the base of dam 6 to 20 ft. into good rock. Two inch diameter grout holes 50 ft. deep were spaced 6 ft. apart along bottom of cutoff trench where the dam was over 200 ft. high and near sides at lesser height holes were placed 10 ft. apart and reduced to a minimum depth of 25 ft. at top of dam. To provide a more gradual transition from solid rock to loose fill, the bottom 10-ft. of 15-ft. layer of rock backing is laid up in concrete mortar. This should minimize

tendency of slab to crack at junction of facing and cutoff wall. Spillway solid rock excavation was crushed to 2½ in. maximum size and sand is obtained by passing crushed rock through a cone crusher. Each 60-ft. panel is completed in one continuous operation, forms being moved while pouring is in progress. Concrete is spouted into place by chutes from side dumping, 2 cu. yd. cars hauled by gasoline locomotive.—H. J. GILKEY

Foundation treatment of the Rodriguez dam on the Tijuana River, Mexico. CHARLES P. WILLIAMS. *Proc. Am. Soc. C. E.*, Aug., 1930, V. 56, No. 6, p. 1291-1297.—This dam of Ambursen type now under construction may be statistically summarized as follows: Height 187 ft. above stream bed, 240 ft. above lowest foundation bed-rock. Crest length 2,000 ft. Buttress spacing 22 ft. center to center. Buttress thickness from 19 in. to 66 in. Deck thickness 25 in. to 64½ in. Deck slope (lower face) 1:1. Buttress slope (downstream face) 1:5. Spillway capacity 150,000 sec.-ft. Purpose: To irrigate 5,000 acres and supply domestic water for Tijuana (population 10,000). Drainage area 1,670 sq. mi. Reservoir capacity 110,000 acre-ft. Estimated possible flood 150,000 sec.-ft. Foundation exploration: 9 borings and 52 test pits. Selection of Ambursen type was due to variability in bearing power of foundation rock and fact that dam crossed a fault plane. Most unique foundation feature was the construction of arch to transfer part of four of buttress loads to better rock. Arch was flat below and underside rested directly on soft rock in central portion. Construction permits redistribution of stresses as required if softer rock yields. In the fault zone concrete cutoff wall was poured in open cut 26 to 33 ft. below original rock surface (66 to 72 ft. below stream bed). Below these levels excavation was continued where necessary through shafts left in concrete.—H. J. GILKEY

Experiences from a power plant construction in India. CARL THULIN. *Teknisk Tidskrift* (Sweden), Oct.-Nov., 1930, No. 11-12, p. 129-135 and 145-152.—The Swedish contracting company "Vattenbyggnadsbyrån" in connection with Rendel, Palmer and Trittion of London, has just completed power plant construction on Malakka peninsula for The Perak River Hydro-Electric Power Co., London. The power is to be used for the fabricating of tin. The construction was started in Dec. of the year 1926. The plant was located 6 miles from nearest road in a dense jungle, so road was constructed first, involving excavation of 143,000 cu. yd., and building of several bridges. Over 100,000 tons of material, sand and stone not included, was hauled over this road. As much as 3,500 tons of cement could be transported per month. Trucks with solid tires lasted only half as long as trucks with air tires, so during the later part all new trucks were supplied with air tires. Working force consisted of 110 Europeans and as many as 4,500 laborers. The Chinese were used where certain skill was required, as carpenters, lumber men, etc. Tamiles were used for hauling and Bengali for the heavy work. Lascars were used for work high up in the air. An amount of 20,000 English pounds was spent on different methods of fighting malaria, with such a good result, that the number of sick people averaged only 2 per cent, and for long periods as low as .5 per cent. Contrary to customary practice these precautions were not started before start of construction. To connect two sides of river a suspension bridge with free span of 450 ft. was built. It was designed to carry a train weighing 20 tons. The dam was 1,380 ft. long with a maximum height of 143 ft. Total concrete 143,000 cu. yd., steel 5,700 tons, and forms 1,080,000 sq. ft. Pouring of concrete was very difficult on account of the heat, during the middle of the day—about 140° F. The Insley System with two towers 300 ft. high and 1,000 ft. apart was used. The quality of sand and stone varied, and it was necessary to establish a regular concrete laboratory. Best mixture was found to be: cement 460, coarse sand 140, medium sand 240, fine sand 120, big stone 400, and small stone 400. The forms consisted of 2-in. planks and could not be reused many times, because they seemed to wear out very quickly.

Some kind of white ants that ate up all kinds of soft wood did considerable damage to the forms and other construction of wood. The power plant was opened on June 28, 1930.—ODD ALBERT

Effect of shrinkage in large concrete dams. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Concrete benches reduce maintenance costs in greenhouses. *Concrete*, Jan., 1931, V. 38, No. 1, p. 33.—Concrete soil benches vary from 200 to 400 ft. in length. Bench consists of two curbs, each 16 in. high, 2 in. wide at top and 4 in. wide at bottom. Aisle side is vertical and inside battered. Two curbs for entire length of bench are cast in one operation in special steel forms. No joints are provided, cracks are immaterial. Other benches consist of precast 1-in. reinforced slabs, 12 in. high. Eight ft. lengths are used for sides and 2 ft. 6 in. lengths for ends. Cast iron angles driven into ground at corners hold slabs in position. Slabs made in steel forms using 1:2 mortar, reinforced with ½-in. galvanized mesh. Sixty slabs are required for each bench.—C. BACHMANN

Application of construction methods with sliding forms. KNEES. *Report 33rd. main meeting German Concrete Association*, March 17, 1930, p. 388-409.—After giving a short historical review of development of concrete and reinforced concrete construction methods, author describes various methods for concreting with movable and sliding forms which are in use in several European countries. System Koethenbuerger is based on working in two zones with alternating use of 2 like form sections. It is especially suitable for bin construction. Similar, but superior in many aspects is system Heine, which was developed in United States. Sliding forms became more in favor in Europe during recent years. Greater uniformity of concrete, better construction control, no danger of separation of aggregates or formation of voids, monolithic structure, savings in form work and easy operation are few of advantages mentioned. System MacDonald, also originated in the United States, is described and a number of silo structures and chimneys show its great field of application. In Holland and Hungary, Spindle method enjoys great appreciation, where storage bins of enormous dimensions were built. High early strength cement was used in most every case. Detailed description is given of construction of a cooling tower, elevator tower in Rotterdam (Holland), clinker silos in Neuwied (Germany), oil seed silo in Emmerich, chimney in Kiel and several cement storage silos showing construction progress with above discussed methods. (cf. Knees, *Zement* (Germany), June, 1930, V. 19, No. 24, p. 561-5; Langenbeck, *Ibid*, Feb., 1930, V. 19, No. 6, p. 127-9; Hoffmann, *Ibid*, Aug. 28, 1930, V. 19, No. 35, p. 828-34; *J. Am. Concr. Inst.*, Sept., 1930, V. 2, No. 1, Abstr. Sect., p. 24: *Ibid*, May, 1930, V. 1, No. 7, Abstr. Sect., p. 102.)—A. E. BEITLICH

Reconstruction of quay system in harbor of Memel. NIEBUHR. *Bautechnik* (Germany), Aug. 15, 1930, V. 8, No. 35, p. 527-8.—Increasing traffic during last years in harbor of Memel demanded extension of quay to 1115 ft. and excavation of harbor to a depth of 26.3 ft. and later to 32.8 ft. Underground consists of large granite rocks embedded in very fine gray marl. Steel sheet piling of Larssen type, 51 ft. long, were driven and rigidly connected with existing quay wall. Concrete filling between both was supported by 30 ft. wooden piles. Storage house of reinforced concrete skeleton construction, 262.5 ft. long and foundations for tracks of unloading traveling crane were carried out in record time.—A. E. BEITLICH

Subway station Frankfurter Allee in Berlin (Germany). *Bautechnik* (Germany), Aug. 1, 1930, V. 8, No. 33, p. 508-10—In construction of about 1.1 mile extension of subway railroad system of Berlin, excavating and scaffolding

were carried out under great difficulties to avoid interference with street traffic. Only 33 ft. width of construction site was available for transportation and mixing of raw materials and distribution of concrete. Two mixers with one yd. capacity each handled total of 78,500 cu. yd. of concrete. Floor of tunnel consists of several layers of reinforced concrete (1:5) with a number of protective coatings of asphalt and bitumen. Gravel was especially prepared and did not contain more than 40 per cent passing 4 mesh sieve and left a residue of at least 30 per cent on 8 mesh sieve. Clay content was below 1.5 per cent.—A. E. BEITLICH

Extension of a quay in the harbor of Reiherwerder (Germany). SCHULZE. *Bautechnik* (Germany), July 25, 1930, V. 8, No. 32, p. 487-91.—Quay is 502 ft. long and supported by one row of flat and three rows of round reinforced concrete piles. Total of 989 piles were driven. Mixture of 1100 lb. blast furnace slag cement and 38.84 cu. ft. gravel with a w/c-ratio of 0.50 was used for manufacture of these 50-ft. piles. Pile heads and ends of reinforcements penetrate about 16 in. into main concrete flat slab of quay. Its bottom layer is made of mixture of 666 lb. iron portland cement with 38.84 cu. ft. gravel, while 550 lb. iron portland cement with 42.37 cu. ft. gravel was used for top layer. Joints between flat concrete piling were filled with fabric hose into which a 1:3 cement mortar was poured. Two coatings with Inertol protect concrete against aggressive waters. Foundations of tracks for traveling ore crane rest on vertical reinforced concrete piles placed in distances of 7.2 ft. Iron portland cement was used exclusively for their construction. During entire construction period all structural materials were carefully examined, also compressive strength of concrete and control of mixing operation and distribution. Moisture of aggregates was determined daily to hold w/c-ratio of 0.50 constant. Concrete was mixed in 17 cu. ft. mixer and distributed from movable platform through wooden chutes.—A. E. BEILTICH

Wolfe's Cove Terminals Construction. C. A. LEIGHTON. *Canadian Engineer*, V. 59, No. 21, Nov. 18, 1930, p. 637.—Wolfe's Cove Terminal is being built by Port of Quebec authorities, City of Quebec, in connection with harbor development in the St. Lawrence River. Project covers river frontage of city proper for a distance of two miles, giving total wharfage of 26,000 ft. Construction consists of quay wall for entire length with piers at intervals, which are 2300 ft. long and 435 ft. wide, extending out into river. Wolfe's Cove Terminal consists of 4300 ft. quay wall made up of timber crib to low water supporting superimposed concrete retaining wall. Cribs are 42 ft. high, 62½ ft. wide at bottom and 30 ft. at top. Concrete wall is 26 ft. high, 19 ft. wide at base and 6 ft. wide at top. Concrete was poured one year after building cribs, to allow time for settlement. Concrete face in contact with water up to mean high tide is faced with pre-cast concrete. Pre-cast units are molded in steel forms and cured with live steam. They are placed in the work after two weeks storage. Concrete mix is 1:1½:3. Concrete backing consists of 1:3:5 concrete of which about 50 per cent is placed through tremie. Top of wall above mean high water consists of 1:2:4 concrete. Quantities required are 6500 cu. yd. pre-cast and 50,000 cu. yd. mass. concrete.—G. M. WILLIAMS

Concrete linings for canals—experiences and improvements. S. KURZMANN. *Bautechnik* (Germany), Sept. 5, 1930, V. 8, No. 38, p. 568-71.—Concrete lining has in general 3 conditions to fulfill. It must (1) protect dams and bottom of canals against attacks of water and provide canal with permanent cross-section, (2) decrease roughness of walls and prevent losses of water and (3) protect adjacent land against excessive water infiltration. Observations were made of behavior of canals of several great Bavarian hydro power stations and experiences and improvements are reported. It was found that expansion joints were widened causing some water loss. No great destruction by frost could be found when concrete was properly placed in not too

lean a mixture. Concrete of a 1:0.7:16 cement-trass mixture had partly failed and was replaced. When water level in canal changes frequently, it is possible that great voids are formed underneath lining by washing away of fine material through joints or small leaks. Collapse of concrete walls results. This can be prevented by placing layer of gravel underneath concrete. Special consideration was given to careful treatment of finished concrete. Sand (0 to 0.2 in.) and gravel (0.2 to 1.2 in.)were used in proportion of 2:3 with 300 lbs. cement for 35.3 cu. ft. concrete. Very uniform and dense concrete was obtained. Surface was kept wet for 5 days and covered with ½ in. boards for 2 days. Then a thin suspension of 1 part of cement and 2 parts of very fine sand in much water was poured over concrete to fill fine haircracks and smoothen surface. Additional information is furnished concerning concreting methods, equipment for such jobs, their capacities, power consumption and economy.—A. E. BEITLICH.

ROADS AND PAVEMENTS

Concrete road construction in Argentine. C. E. REYNOLDS. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 691-697.—In describing building of concrete roads in Argentine following topics are treated: The design of roads including the shape of sections used and methods of construction; specifications for materials, including size of ingredients, proportions used, and strength of material; mixing and placing of concrete; method of control of water content by "slump tests"; description of the types of expansion joints used; details of method used in reinforcing concrete.—JOSEPH MARIN

Settled concrete road slabs lifted and leveled by force pump. *Concrete*, Jan., 1931, V. 38, No. 1, p. 27.—Settled concrete slabs are being raised and leveled by pumping mixture of loam, cement and water under them. Pump has chamber in which materials are mixed. Mixture is forced through 2-in. hose. Holes slightly smaller than outside diameter of the hose are bored in the settled pavement on about 6-ft. centers. Hose is forced into one of holes and pump started. Swelling of hose by pressure of mixture holds end fast in hole. Pressure is 5 lbs. or less per sq. in. Mixture hardens in 10 to 12 hours.—C. BACHMANN

SHOP MANUFACTURE

Piece-work vs. day labor. *Concrete Products*, Dec., 1930, V. 30, No. 6, p. 34-35.—This symposium was participated in by 62 readers of whom 14 or 22½ per cent are using the piece-work system in some form or other. Its customary application is only to the men who are operating machines producing block, roofing tile, etc., but occasionally it is applied to other men throughout the plant.—E. S. HANSON

An excellent vibrating apparatus, the table vibrant. JACQUES DUPY. *Rev. materiaux construction trav. publics* (France), Nov., 1930, No. 254, p. 458-9.—An apparatus for mechanical vibration of concrete products requiring small power. The amplitude of vibration can be regulated. Machines of 880 and 4,400 lb. capacity are described.—F. O. ANDEREGG

Gelatine moulds. *Conc. Building Conc. Products* (England), Dec., 1930, V. 5, No. 12, p. 165.—In preparation of the gelatine for casting concrete figures, gelatine sheets should, if new, first be placed for 15 minutes between wet sacking to make them pliable. They should then be heated in a steam jacketted pan until the gelatine is just hot enough to pour, usually after about six hours, gentle heating. All models should first be varnished with shellac varnish and oiled with colza oil. The moulding may be used for about ten times. For reuse it should be washed and cut into 1-in. cubes and melted, without the addition of water, in the pan. New gelatine may be added to freshen up the old.—JOHN E. ADAMS

Operator utilizes excess pea gravel in manufacture of concrete blocks. *Pit and Quarry*, Dec. 3, 1930, V. 21, No. 5, p. 46-48.—Simple outdoor equipment has been installed by Shera Sand & Gravel Co., at its 300-ton plant near Richmond, Ind. A survey indicated possibility of selling locally as many concrete blocks as usual plant crew could produce during the usual idle periods. This has proved correct and program will be continued as it assures year-round employment. Operations include excavation, screening and washing of sand and gravel. Block machine produces 35 blocks per hr., each 8 in. by 8 in. Concrete is prepared in a worm-type continuous mixer. During past summer stock of pea gravel was maintained at minimum by this practice.—A. J. HOSKIN

Material producers making pipe. DUDLEY W. MOORE. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 89.—New plant of the Oklahoma Cement Pipe Co., Tulsa, Okla., is owned by Price Sand Co. and Hughes Stone Co., is also connected with Builders Concrete Co., which sells ready mixed concrete. The new pipe plant acts as an auxilliary plant for ready mixed concrete when this is desirable. Pipe plant produces 60 to 70 tons per day, largely storm sewer pipe handled by 2½-ton electric crane. An 800-ton octagon steel storage tank holds pipe materials. It has five compartments for stone and sand and cement compartment of underslung type, all fed by single elevator supplied from track hopper by an apron feeder. Bins are discharged by gravity to a 7-yd. weighing batcher. Batched material goes by conveyor system to a 21-cu. ft. mixer and heavy duty pipe making machine. Water is added from a measuring tank. Pipe is molded on a revolving table and then core is pulled by small electric hoist and molded pipe is sent to the curing room. Sizes are from 12 in. to 60 in. dia., sizes above 18 in. being reinforced. Curing is by placing pipe under water sprays for five days after which it is stored in the yard. All machines have individual motors with inclosed gear reducing units.—EDMUND SHAW

New concrete brick factory in Surrey. *Conc. Building Conc. Products* (England), Nov., 1930, V. 5, No. 11, p. 144.—An up-to-date factory for the manufacture of concrete bricks, is described, where bricks of all standard shades are made. Only mineral oxides are used as coloring agents, so there is no risk of fading. For partition walls, bricks are mixed 10:1 while 8:1 mix is used for external wall bricks. For greater strength 4:1 or 3:1 bricks are made. These bricks can be easily cut in any plane, and although they prevent penetration of moisture, they do not cause condensation. The materials are raised by a pump through a pipe line to three large settling bins beside the supplying lake, and to assist the raising, charges are exploded below water twice weekly. From the bins the sand is taken by jubilee skips to factory. Sand is cream colored, while cement and coloring material are ground together in ball mills specially designed. The proportions for the various colors are as follows: Red, 8 per cent of brick red oxide of iron by weight of cement; dark red, 8 per cent crimson red oxide; dark brown, 4 per cent crimson oxide plus 2 per cent mineral black; light brown, 4 per cent brick red plus 1 per cent mineral black; yellow, 5 per cent yellow ochre; tan or biscuit, 1¼ per cent brick red oxide; light stone, 2 per cent yellow ochre. The mixers used are of the pan type with two rollers which just clear the bottom of the pan, of 4 or 5 ft. capacity and turn out each about 1200 bricks per hour. The mixture is only sufficiently wet to bind if squeezed in the hands. From the mixer the materials are elevated to hopper over a revolving cylinder carrying three brick molds into each of which correct amount of concrete is rammed. Pallets are passed from discharge end of machine to steel racks and thence to curing shed. Bricks are kept moist for 48 hrs. At first they are sprayed with moisture to avoid pitting, then racks are covered and well saturated for rest of the 48 hours. For transporting to covered storage, portable roller conveyors with adjustable stands are used.—JOHN E. ADAMS

Materials

Aggregates

Two new gravel producing plants in Milwaukee district. Gordon F. Daggett. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 52.—Success with two sand and gravel plants in district well supplied with plants before they were built is ascribed to making specially prepared aggregates. At plant of Kohler Bros., Prospect Hill, Wis., sand was not washed originally as it was thought to be clean enough. Later both sand and gravel had to be washed to meet specifications, the sand by means of a double screw washer and classifier. At Central Sand and Gravel plant, Milwaukee, Wis., special washed sand was made by first passing sand through steel cylinder with lifting plates and baffle rings. After scrubbing in this cylinder everything is passed to a twin screw classifier and dewaterer. The washed product is stockpiled by belt conveyor.—Edmund Shaw

Preliminary tests on effect of size of aggregates on strength of concrete. C. E. Proudley. *Nat. Sand Gravel Bul.*, Dec., 1930, V. 11, No. 12, p. 13-18.—The laboratory of the National Sand and Gravel Association has carried out some preliminary tests in an effort to obtain some definite information on effect of size of aggregates on strength of concrete. One grading of sand and four gradings of gravel were used. The tests were made under the following conditions: The gravels, having different maximum sizes, were graded from fine to coarse in a similar manner; the sands and cement were of constant physical characteristics; and the consistency of the concrete as measured by flow table was same for all aggregates and proportions. While their principal value has been to point out some of requirements for more comprehensive investigation to be conducted in the future, following conclusions may be drawn from a study of results of these preliminary tests: (1) The coarser sizes of gravel gave higher strengths in compression and modulus of rupture than smaller sizes. For the average of proportions studied, the ¾ in. maximum size of gravel was about 12 per cent higher in compression and 7 per cent higher in transverse strength than the ⅜ in. maximum size. The 2 in. maximum size was about 11 per cent higher in compression and 8 per cent in transverse strength than the ¾ in. maximum size. (2) The richer mix, which also contained a higher proportion of sand, was less affected by changes in maximum size of aggregate than the leaner ones. (3) Test results showed fairly uniform relations between strength and water-cement ratio and between strength and void-cement ratio. They indicate that effects of changes in size of coarse aggregate were due to their influence on the quantity of mixing water required to produce uniform workability. (4) Coarse aggregates of different maximum sizes require different quantities of mortar to produce comparable workabilities. Accurate control of this factor is necessary in order to obtain proper comparisons. Further tests with particular attention to this factor are required to permit of more definite conclusions concerning effect of maximum size of coarse aggregate.—P. McKim

Dual washing system at new Clarence plant. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 33.—The new plant of the Clarence Sand Co., near Alexandria, New York, has a dual washing and screening system. The excavation is by 3-yd. scraper pulled by a 150 h. p. motor. This discharges to a hopper with a 30-in. belt conveyor feeder. A 24-in. belt conveyor takes the material to a 48-in. by 8-ft. vibrating screen with 2¼-in. mesh screen above and ¼-in. below. The oversize goes to 26-in. jaw crusher; intermediate sizes to double screw washer; the sand to the sump of an 8-in. pump. This pumps at about 80 per cent solids and 20 per cent water through a 6-in. pipe and it takes the place of an installation in which the sand was dewatered by drags and by screws and then carried by a belt conveyor. The washed gravel (with

crusher product added) goes to secondary washing plant above the bins. The gravel is rinsed and sized by vibrating screens and the sand is classified by a classifier with automatic discharge. The fine sands in the classifier overflow are recovered in automatic settling tanks.—EDMUND SHAW

Young men build one crusher stone plant. RAY F. SCHNEIDER. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 48.—At plant of Bussen Quarries, Jefferson Barracks, Miss., mining has superseded quarrying, the overburden being from 10 to 30 ft. Another quarry has been opened to be worked by an open pit. This will be used to supply stone in good weather and mining will be practiced in cold and bad weather. The stone is carried from the quarry face to the plant in 1¾-ton and 5-ton trucks. Entire crushing equipment is one "Jumbo" hammermill, which can reduce large rocks to 2 in. and finer in one operation. Feed opening is 36 x 42 in., fed by apron feeder. Crushed material goes by elevator to revolving screen, making four products. Oversize goes back to the crusher by a conveyor belt and chute. The minus ¾ in. goes to vibrating screen and other products directly to bin. Mixing conveyors below bins permit furnishing any desired combination of sizes.—EDMUND SHAW

Control and improvement of gauging of sand, gravel and river pebbles for concrete. A. JANOUSEK. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—It is the exception for gravel taken from river to correspond with rational requirements for concrete. One of most important objects of control is a granimetric analysis, as accurate as possible, promoting practical and economical improvement of aggregate, and determining what conditions govern strength and other required qualities of concrete. Chief points are: (1) Determination of sizes of stones and grains, their geometric shape, and importance to be attached to them as regards strength of concrete. (2) Determination of individual and total surfaces of grains, and influence of this on strength of concrete. (3) Determination of quantity of grains having equal dimensions, and relative proportions. (4) Application of these results to improve economically gravel as obtained. Usual procedure consists of successive siftings of natural mixture. Another way is to measure dimensions of grains directly, which, however, can be done only to a small quantity. Superficial area of grains can be found only approximately by experimental methods, or by calculation based on results of sifting. To make practical and economical use of these results a continuous control of works is necessary, and parallel with this control, a system of regulating mixture which must operate as automatically as possible and be independent of any initiative on the part of unqualified persons. There is still no satisfactory experimental method for determining exact surface area of stones. Effort has been made to give some indications on the subject of calculating this area, which is of great importance, especially in regard to tensile strength of concrete. Further, the author has examined important question of international unified methods of control, arriving at proposal brought forward in concluding paragraphs of paper; the gist of the matter being a series of screens answering both to scientific and to practical requirements, and suggestion being screens with circular holes .039, .118, .285, .590, 1.180 in., or .039, .079, .197, .394, .787 in. in diameter. Finally, very limited possibilities of using fineness modulus is dealt with and simple method of comparison of granimetric curves and of correcting random gravel is suggested.—EDITIONS LA TECHNIQUE DES TRAVAUX

Slag aggregate follows same course as others. *Rock Products*, Jan. 3, 1931, V. 34, No. 1, p. 144.—Production and sale of crushed slag shrunk 8 to 10 per cent in 1930. Prices were about same as in 1929. Even with present plants operating on 50 per cent capacity basis as indicated, several new plants were built during past year and many older plants were remodeled to operate

more economically. Employes of slag industry worked 75 per cent of time compared to 1929, with an indicated reduction of 25 per cent since 1928. Prices for 1931 are expected to remain at their present level with tonnages showing an increase, especially during the latter half of year. No changes are expected in employes' wages nor will there be an appreciable change in the number of men employed. Important new plants include Buffalo Slag Co., The Duquesne Slag Co., and Cashion Slag Co., the latter notable for simplicity of flow of material through plant. Birmingham Slag Co. is rebuilding its plant at Ensley, Ala., to have capacity of 2,000 lbs. National Slag Association opened a new research laboratory. In the JOURNAL of the Concrete Institute Proceedings, C. A. Hughes and A. S. Levens give some interesting information relative to the use of pulverized slag as a puzzuolanic material. In their tests, two kinds of volcanic ash, a blast furnace slag and burnt shale were used as puzzuolanic material and comparisons of compressive strength, shrinkage, etc., noted. (cf. A. C. I. JOURNAL, Dec., 1930; *Proceedings*, V. 27, p. 317). On compression tests in general, slag showed the greatest puzzuolanic activity and greatest strength. Shrinkage increased with increase in effective water to cement plus puzzuolana, slag being an exception to this rule.—EDMUND SHAW

Composition of mixing and application of concrete in engineering works. C. J. JACKAMAN. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—This paper, from point of view of a civil engineering contractor, endeavors to record outstanding features which might differentiate concrete works undertaken in England from those in other countries where different conditions prevail. Composition: Aggregates.—More readily available materials for these may be classified as: (a) for southern and southeastern parts of England (with some isolated areas elsewhere), natural river gravels and large areas of suitable sea beach gravels; (b) for districts in the southwestern, western, north midland, and northern parts, various igneous rocks. The former are in great demand and are more readily prepared for use as aggregate and hence in general prove economical; latter class of rocks requiring much more expensive production and preparation have not been so fully exploited. Preparation of aggregates has formed separate and increasingly important branch of concrete industry adjacent to large towns. For works of magnitude, however, every phase of preparation of raw material is commonly undertaken by the contractor on the works site, there being available power-driven machines for various operations of breaking natural rock to acceptable sizes, cleaning, screening and grading. Matrices. —Portland cement is preeminent among all other materials for matrix of concrete work. Improvement of its quality has been so continuous as to have almost entirely out-distanced use of other materials for the purpose. Rapid hardening varieties of portland cement have attained great popularity, justified by advantages they offer in many kinds of work. 1. Proportion of concrete. —For constructional works in England it is still invariable practice of engineer to assume responsibility for specifying proportions of constituent materials and usually maximum and minimum sizes for aggregates and sand. Determination of exact ratios of intermediate grading between limits assigned has not yet been generally adopted. Proportion is still almost invariably stated by measure of volume for coarse and fine aggregate; but for portland cement greater exactitude is sought by proportioning by weight. Little progress has been made in adoption of any method for relating proportion of mixing water to quantity of cement. Due check on quantity of water used is, however, maintained by increasing adoption of slump test of concrete after mixing. 2. Mixing concrete.—Various types of mechanical concrete mixers in general use are noted. "Batch" or non-continuous mixer, is most generally favored for securing uniformity of quality in product. Internal-combustion engines with petrol fuel have outstripped all competitors for motive power for these machines. 3. Distribution of concrete.—English engineers have shown great reserve in permitting use of automatic distributing plant and this attitude is receiving

support from importance now assigned to concrete having minimum water content consistent with easy manipulation. 4. Deposition of concrete.—No generally applicable, economical, and equally satisfactory method of consolidating concrete, when placed, has yet been produced to compare with carefully supervised manual effort. 5. Formwork.—Use of timber for formwork has not yet been challenged by any serious rival, owing to high initial cost of steel forms which forbids their adoption on any but large expansions of repetition work.—EDITIONS LA TECHNIQUE DES TRAVAUX

CEMENT

Studies on mixed portland cements—Part 6. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, p. 279-85. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1191.—Neo-Soliditit cement was made from slate mixtures. Soluble portion of slate was determined by successive treatments with sodium hydroxide solution (10 per cent) and hydrochloric acid (5 per cent). Mixtures of 55 to 85 per cent clinker and 15 to 45 per cent slate gave surprisingly high strength data, higher than pure cements.—A. E. BEITLICH

Studies on mixed portland cements—Part 7. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental binding, No. 6, p. 192-6B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Expansion and contraction of several kinds of cements: portland, blast furnace slag, high alumina and soliditit cements (to which certain colloidal materials were mixed in order to correspond with Japanese Neo-Soliditit) were studied. Specimens were heated from 392° to 1472° F. Great shrinkage led to complete destruction of several cement samples. High alumina cement showed greatest resistance. Portland cement shows a considerable increase of free lime content under this treatment. Same increase can be observed between 752° and 1112° F. in the case of soliditit cement and blast furnace slag cement, while free lime content drops sharply when heated from 1472° to 1832° F. A combination of free lime with acid constituents of colloidal materials seems to take place at such high temperatures. (cf. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, p. 279-85.)—A. E. BEITLICH

Special cements and concretes. A. HASCH. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Maximum properties of cement and concrete can be considered from the general physical, mechanical and chemical points of view. The comparison of high early strength portland cement with Ciment Fondu (aluminous cement) shows that the latter is principally used on account of its greater capacity of resistance, as the high early strength of portland and cool setting cements is sufficient for majority of constructional requirements. Also they can be used for concreting at low temperatures. Statistical consideration of experimental results using Bernouilli law of the greatest number appears also to be an important method in future work of material testing.—EDITIONS LA TECHNIQUE DES TRAVAUX

Further developments in closed circuit dry grinding. H. G. WRIGHT. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 46.—Micro-photographs show that products made by reverse current air separation are more angular and therefore have more surface exposed than unclassified products. Hence they should burn more easily. Air classified material is shown to carry less oversize. That which makes good burning in kilns is not amount reduced below 200-mesh but that which is below 100-mesh. Tables are given comparing discharge of tube mills by gravity and air classified reverse current system. These show a less amount retained on 100-mesh for air classified products for pro-

portionately same grinding through 200-mesh. (cf. *Rock Products*, July 19, 1930, V. 33, No. 15.)—EDMUND SHAW

Examination of refractory materials for cement kilns. J. SCHAEFER. *Zement* (Germany), Nov. 20, 1930, V. 19, No. 47, p. 1112-3.—Durability of brick lining of rotary kilns depends largely on structure of bricks (shape and amount of voids and gradation), viscosity and chemical composition of clinker, reaction temperature, composition of bricks and duration of reaction. An improved method is described for testing of bricks under conditions which are similar to those encountered in actual mill practice. Bricks are heated in a high temperature furnace to different test temperatures and a powerful gas flame carrying raw mixture is blown against them. Effect of heat and reaction of clinker particles on brick surface can be easily determined. Also effect of great temperature changes on brick can be studied by rapid removal of brick from furnace. Chemical analysis of top layer shows depth of penetration and changes in composition. Photographs show different behavior of ordinary refractory brick, Dynamidon brick and Magnesidon brick.—A. E. BEITLICH

The grinding of cement and porosity of concrete. A. POULSON. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—No concrete is solid, and to secure greatest protection against chemical action (including that due to infiltration of pure water) a puzzolanic cement must be used to neutralize deleterious lime which will then unite with hydraulic constituents of puzzolana to form compounds like those in cement itself. In addition the "hydraulic index" should be tested and ought not to be less than ⅞; also mechanical strength. Reference is made to previous publications in respect of rational use of puzzolanas, and of their examination: 1912: New York Congress on Materials: *Cement in Seawater*. 1923: London Congress on Navigation; No. 53: *Portland cement in seawater and chemical value of puzzolanas*. 1924: Cairo Congress on Navigation: *Hydraulic mortar in seawater*. 1929: For Zurich Congress on Materials, 1931: *Compactness and chemical resistance of concrete*.—EDITIONS LA TECHNIQUE DES TRAVAUX

Fused cement and its influence in industry. EMIL STOCK. *Osterr. Bauzeitung* (Austria), Nov. 15, 1930, V. 6, No. 46, p. 752.—Resume in German of Italian article dealing with development of high alumina cement industry. Fused cements are used with advantage (1) for jobs which must be finished in a short time, (2) for winter construction and (3) for structures which are exposed to chemical actions or aggressive solutions. Rapid development of this type of cement led to increased efforts to improve ordinary portland cement and to study its physical and chemical properties on a scientific basis. (cf. *L'Industria Italiana del Cemento* (Italy), 1930, No. 5.)—A. E. BEITLICH

Researches on the rotary kiln in cement manufacture—Part 9. GEOFFREY MARTIN. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 54.—Heat required to decompose $CaCO_3$ at different temperatures is determined. This is usually taken to be 774 B. t. u. per lb. at ordinary temperatures. But calcite, raw material of cement, takes 756 B. t. u. per lb. From researches of several investigators, especially John Johnstone, a formula is worked out and steps are given in detail. From formula a table has been made giving the heat of disassociation of $CaCo_3$ in the rotary kiln. This shows that, beginning at ordinary temperatures, required heat is about 756 B. t. u. for calcite, but at 1443° F., (the lower limit of decomposition in rotary kilns) it is 685 B. t. u. At 1481° F. (upper limit of initial decomposition), it is 681.6 B. t. u. With increasing temperatures B. t. u. required decrease, dropping from 644.2 at 1832° F. to 341.1 at 3632° F. At 4940° F. no heat is required but there is evolution of 1.25 B. t. u. At 5432° F. there would be an evolution of 156.1 B. t. u. It would be impossible to secure this because of enormous pressure that would be required to keep $CaCO_3$ from splitting into CaO and CO_2 before reaching critical temperature of 4940° F.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture—Part 10. GEOFFREY MARTIN. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 49.—In calculating amount of heat absorbed in making one pound of portland cement clinker, author assumes clinker containing 64 per cent CaO, 7 per cent Al_2O_3, 3 per cent Fe_2O_3, 22 per cent SiO_2 and 4 per cent non-essentials. To make 100 lb. there will be required 119.5 lb. $CaCO_3$, 18.46 lb. kaolin, 15.40 lb. hydrated silica, and 3.12 lb. ferric oxide. A study is made of each of these showing quantities of heat required to dehydrate, when this is necessary, and to raise material to temperature required to make clinker. The final quantity of heat required for 100 lb. clinker figures to 81,192 B. t. u. for decomposing $CaCO_3$, less 17,993 B. t. u. evolved in exothermic reactions, plus 28,665 B. t. u. absorbed in raising 100 lb. of clinker from 1481° F. to 2498° F., a net total being 91,864 B. t. u. Conclusion is that it should be carefully noted that it is fallacy to state total minimum quantity of heat required to make 1 lb. of clinker is 918.6 ÷ 652.5 = 1571.1 B. t. u., because this statement takes no account of quantity of heat recoverable from hot clinker and evolved gases. In making 1 lb. clinker raw material mixture must be subjected to steadily increasing temperature starting at 60° F. and finishing at about 2,498° F., and raw mixture must absorb 652.5 B. t. u. below 1,481° F. and 918.6 B. t. u. above 1,481° F. before 1 lb. clinker can be formed.—EDMUND SHAW

Studies on calcium ferrites and iron cements—Part 3 and 4. SHOICHIRO NAGAI AND KATSUHIKO ASAOKA. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental Binding No. 6 and 7, p. 190-2B, 256-9B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1191.—New data are presented of this comprehensive investigation (cf. *Kogyo Kwagaku Zasshi* (Japan), May, 1930, V. 33, Supplemental Binding No. 5, p. 161-4; JOURN. Amer. Concr. Inst., Dec., 1930, V. 2, No. 4, *Abstracts*, p. 96). Dicalcium ferrite, $2CaO.Fe_2O_3$, reacts with water in formation of calcium hydroxide which produces its strength properties. These decrease in water storage and stay fairly constant in air storage. Constancy of volume of dicalcium ferrite mortar was studied by Bauschinger and Le Chatelier methods and compared with properties of portland cement and Kuhl cement. When mixtures of SiO_2, Fe_2O_3 and CaO are fused between 2552° and 2732° F., calcium ferrite is formed first, then combination of CaO and SiO_2 to di- and tricalcium silicate takes place. Synthetic mixtures in the field SiO_2—Al_2O_3—Fe_2O_5—CaO were prepared by replacing 1 part of ferric oxide (8-9 per cent) by alumina (2-3 per cent) at temperatures between 2552° and 2732° F. Cements consisted of about 15 to 16 per cent dicalcium ferrite, 6 to 7 per cent tricalcium aluminate and various amounts of di- and tricalcium silicate. Formation of tricalcium silicate was only possible, when ferrite or aluminate were present.—A. E. BEITLICH

Studies on calcium ferrites and iron cements—Part 5. SHOICHIRO NAGAI AND KATSUHIKO ASAOKA. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental Binding, No. 8, p. 312-5B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Temperature of combination of iron cements lies above sintering temperature of ordinary portland cement due to small amount of calcium aluminates. These temperatures are between 2642° and 2732° F. Chemical composition is about 65.2 per cent CaO, 23.3 per cent SiO_2, 2.50 per cent Al_2O_3 and 9.00 per cent Fe_2O_3 and their constitution is approximately 52.2-54.5 per cent tricalcium silicate, 20.2-23.7 per cent dicalcium silicate, 3.5-5.8 per cent tricalcium aluminate and 14.8-16.0 per cent dicalcium ferrite. Molecular ratio between CaO and SiO_2 is between 2.62 and 2.67. Strength data obtained by small-piece testing methods are lower than ordinary portland cement strength.—A. E. BEITLICH

Properties of Concrete

The ideal gauging of concrete mixes. H. Imberechts. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The object of the researches dealt with in this paper is to establish a method of calculating proportions for concrete mixes based upon knowledge of curves of granimetric composition for elements involved. Investigation starts with the Fuller-Bolomey curve, $P = A + (100 - A)\sqrt{d/D}$. After determining best value for constant A, percentage of fine elements, including cement, not exceeding 0.02 in. size, which corresponds to the strongest concrete—for a given water content—still easily workable and of satisfactory appearance when fractured, researches were extended to cover all acceptable mixtures between two limits. The lower limit was introduced to secure concrete still easily workable into place, the upper limit to restrict falling off in strength. Reason for laying down these limits is impossibility of making particular cases conform precisely to a given "typical composition"; it is necessary to allow certain tolerances, and curves of "limiting composition" furnish tolerances which can be prescribed.—Editions La Technique des Travaux

Control of concrete construction of the German railroad concern (Reichsbahn). Vogeler. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 236-61.—After discussing contents of recently issued German specifications for mortars and concrete, author describes field control outfit and equipment for stationary concrete control laboratories. All materials: cement, lime, trass, aggregates and mixing water are submitted to careful examination. Concrete is tested for its tensile, compressive and flexural strength, water permeability and resistance against aggressive solutions. Furthermore, admixtures, water proofing materials, protective coatings and bricks are examined. Detailed description is given of equipment and machinery for strength and other tests. Special consideration is given to sieve analysis and exact proportioning of aggregate gradings. Samples of good concrete and concrete that failed, with necessary explanations facilitate understanding of specifications. Progress during first year is very satisfying. Program for second year includes examination of methods and materials for water tightening of concrete foundations, protective coatings, plasters and surface finishing. These will be studied especially on abutments, supporting walls, piers, bridges, culverts and tunnels.—A. E. Beitlich

Deterioration of concrete in hydraulic structures. A. Ekwall. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Inspections of hydraulic structures in Sweden displayed some defects in concrete 10 years old. Continued observations proved, that concrete in some cases was subject to increasing deterioration, apparently due to qualities of concrete materials, to methods of making concrete or to conditions of exposure. Investigations above described were arranged to ascertain causes of such increasing deterioration. It had been observed that deterioration occurred only in structures exposed to water on one side and was caused by solubility of lime and decomposition of other chemical combinations in hydrated cement. Particular attention was therefore placed on solubility of cement and aggressiveness of water. In combination with investigation relating to these factors, practical experiments have been made with concrete exposed to low water pressure for several years. Investigation proved that Swedish natural waters need not be considered aggressive in this respect. Importance of using sand, relatively free from organic substances, was impressively established. Leaner mixtures than 1:4½ should not be used for concrete exposed to water pressure from one side. Replacing cement by an admixture of 10 per cent lime or 5 per cent calcium chloride improved watertightness,

while 25 per cent trass or slate powder gave practically no improvement. - EDITIONS LA TECHNIQUE DES TEAVAUX

Investigation of the behaviour of cement mortar in hot water. "Versuche ueber das verhalten von zementmoertel in heissem wasser." OTTO GRAF. Report of the Institute for Testing Materials at the Technische Hochschule in Stuttgart (Germany), during the years 1928-1929; Publication No. 62 of German Committee for reinforced concrete. Berlin 1930, Wilhelm Ernst und Sohn, R. M. 4.30. Reviewed in *Bautechnik* (Germany), Dec. 5, 1930, V. 8, No. 52, p. 770.—Test pieces of portland cement, high early strength cement and high alumina cement were exposed to water of 68, 122 and 194° F. Permanent storage of testpieces in hot water (194° F.) resulted in an increase in strength of lean mixtures (1:6) while rich mixtures (1:3) lost some of their strength. Behavior in water of 122° F. was varying according to different kinds of cement. Frequent rapid changes of temperature led to considerable reductions in strength and were different with different cements. Conclusions are drawn concerning use of such cements for special structures (hot water tanks).—A. E. BEITLICH

The effect of temperature on flat arches constructed of reinforced concrete. CHARLES S. GRAY. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—First part studies temperature range which may be expected in an arch, variation of temperature throughout the bridge and ratio of internal to external temperature. Second part treats three hinged arches and considers movement of crown hinge and change in horizontal thrust for a temperature variation of 30° F. Third division considers arches of two hinges; curves and tables are given showing variation of temperature thrust and temperature moment at all points of span for various ratios of rise to span for any temperature range. In fourth section are treated arches without hinges; curves are given to show variation of temperature thrust and temperature moment at all points in span for various ratios of rise to span. Fifth section considers effect of change of shape of arch axis upon temperature thrusts and moments.—EDITIONS LA TECHNIQUE DES TRAVAUX

Experiments with columns reinforced with steels of high strength properties. R. SALIGER. *Osterr. Bauzeitung* (Austria), Nov. 1, 1930, V. 6, No. 44, p. 723-30.—Progress of joint investigation of a German and an Austrian institute for testing materials on the subject of reinforcing of concrete columns is reported. Tests with steels of high quality are finished and results reported. Eight different types of columns with octagonal cross-section and circumferential reinforcements of 0.78-in. hoop irons or 0.2-in. round steel were broken. Longitudinal reinforcements consist of 4, 6 or 8 steel rods, 1.18 in. and 1.65 in. thick. Diameter of columns is 13.8 in., their length is 4 and 9.8 ft. Tests in Vienna were made with columns, reinforcements of which were welded in center. Tables show changes in form of columns when exposed to pressures from 10 to 200 tons. Under high pressures, uniformity between column core and outer layer is destroyed. Columns of 9.8 ft. length show a 15 per cent lower compressive strength than 4-ft. columns. Columns with welded reinforcements are of no advantage. Results of tests are carefully analyzed and conclusions are drawn for application in practice.—A. E. BEITLICH

Changes in the action and strength of hyperstatic reinforced concrete structures with time. H. LOSSIER. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Concrete undergoes with time a number of changes modifying action of reinforced concrete structures both from point of view of elasticity and of strength. The former changes, in structures which are hyperstatic, affect sharing loads between the different members. Although generally speaking it does not appear

a priori that these changes are of such nature as to affect safety of the works, yet it is important that question should be studied and checked by experiments in hope of arriving at a solution.—EDITIONS LA TECHNIQUE DES TRAVAUX

Measurement of temperature in concrete during its hardening. F. G. PEREZ. *Ingenieria* (Mexico), Nov., 1930, V. 4, No. 11, p. 418-420.—Small cracks in the buttresses of the Rodriquez Dam (Ambursen Type) in lower California were investigated by a group of consulting engineers who concluded that temperature developed in interior of mass during hardening was one of the causes. This led to investigation of effect of fineness of cement on temperature produced, the data from which is shown graphically in the report. The investigation compared a cement ground to 94 to 96 per cent passing a 200-mesh sieve with one that had 84 to 86 per cent passing that sieve. Temperature measurements were made electrically with resistance type pyrometers inserted in the concrete at five points. Readings were taken at short intervals during first day and at increasing intervals thereafter until they became practically constant. Temperature during setting of concrete made of finely ground cement reached 78° F. while air was 67° F. Concrete made of more coarsely ground cement reached 71° F. while air was 53° F. Temperature of concrete ascended for three days for finely ground and four days for coarsely ground cement. Maximum temperature attained in the concrete with finely ground cement was 126° F. and 118° F. with the coarsely ground cement. Both kinds of cement maintained a temperature higher than surrounding air for two weeks. Maximum temperatures were obtained within four ft. of the exposed side of concrete.—C. G. CLAIR AND M. N. CLAIR

Report on the Swedish investigations and experiences concerning shrinkage and temperature in concrete. N. ROYEN. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Great variation in temperature (95° F. to —56° F.) existing in some parts of Sweden have made it necessary to pay particular attention to such factors as heat generated by setting and hardening of concrete and the influence of temperature and shrinkage. Investigations have been made to find relation between temperature of concrete and air temperature for different slab thicknesses. Heat generated during setting has been investigated by experiments for a number of different brands of cement. Amount of high alumina cement required to obtain, by heat generation, a sufficiently high temperature for curing concrete made at temperatures below 32° F. and effect of low temperature on hardened concrete have been objects of other investigations. Results of experiments on shrinkage and swelling of concrete are given, and finally are stated some measures taken in design and construction of dams in northern Sweden to decrease effect of temperature and shrinkage.—EDITIONS LA TECHNIQUE DES TRAVAUX

The consistency of cast stone. H. A. HOLT. *Conc. Building Conc. Products* (England), Jan. 1931, V. 6, No. 1, p. 2.—The difficulty of measuring consistency is pointed out, since slump tests are for use with much wetter mixes than are encountered in the manufacture of cast stone. "Dry" refers to a mortar which will ball in the hand and leave it clean, but will break and crumble when dropped. It may be mechanically tamped in a mold without the appearance of moisture on the surface, and without becoming plastic. "Semi-dry" mortars will ball in the hand and leave it clean, but when thoroughly tamped in a mold, will become just plastic but no more, and water can just be seen on surface. "Plastic" mortar will just moisten the hand and will ball, barely fails to slump, and will not flow down an inclined plane, but when tamped becomes a stiff quaky mass with some moisture on the surface. For strongest, densest and most impermeable concrete minimum amount of water required to produce plasticity only, must be used. In case of 1:2:4 concrete, 6 per cent water by weight of total dry mixed materials would thus be required.

In many methods of manufacture of cast stone, wet mixes are unavoidable. In case of spun concrete moisture is brought to surface by centrifugal action and is removed before concrete has hardened. On account of excessive labor cost, dry mixes are only practicable with mechanical tamping or consolidation. When little mixing water is used proper curing is essential for case hardening the concrete, and completing hydration of cement on the surface. Experiments show that maximum temperature accompanied by saturated humidity produces maximum strength in minimum time. For surfaces which are to be tooled or brought to an ashlar finish, fine sand and semi-dry or dry mixes are recommended. Wet mixes give better results when coarser aggregates are used, and polished or scrubbed surfaces are required. Plastic concrete tends to produce less crazing and greater adhesion of cement to reinforcement, less risk of disintegration by frost or acid atmospheres than dry and semi-dry mixes, but is less speedy in production.—JOHN E. ADAMS

The grinding of cement and porosity of concrete. (See MATERIALS—CEMENT)

ENGINEERING DESIGN

BUILDINGS

New construction of a recovery home for children in Koenigsfeld (Germany). ALFRED FISCHER. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1092-3.—Building is made as homogeneous reinforced concrete skeleton structure and satisfies all demands of modern hygiene. Cantilevered construction of second floor forms roof above dry, open air playground and is supported by reinforced concrete columns.—A. E. BEITLICH

Progress of experimental statics of rigid frames. F. KANN. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Method developed by author for experimental solution of frames with a high degree of rigidity rests on direct measurement of rotation of joints and bars of a system elastically deformed. In this purpose he has made use of a novel manner of constructing model of bars of celluloid joined at corners by blocks of hard wood which permits of production of very considerable deformations. By design of elastic member and measurement of angles of tangents and chords we can measure exactly rotations, which allows of calculation of moments at joints or their ratios useful for completing or controlling the calculations. Comparison has been made for frames without struts at angles and for frames with struts reaching to 1/5 of length of beams and uprights. Measured angles agreed closely with calculations when taking action of struts into account. Use and development of method in practice for systems with a high degree of rigidity is therefore very promising.—EDITIONS LA TECHNIQUE DES TRAVAUX

Collapses of reinforced concrete structures. LUIGI SANTARELLA. *Osterr. Bauzeitung* (Austria), Nov. 22, 1930, V. 6, No. 47, p. 764—Reasons for collapses of reinforced concrete structures are classified in 4 groups: (1) errors in calculation, design or construction, (2) impracticable construction or erroneous reading of blueprints, (3) bad condition of structural materials or incorrect mixtures and (4) too early removal of forms and lack of protection against unexpected damages. Directions to remedy such faults are given. (cf. *L'industria Italiana del Cemento* (Italy), June 1930.)—A. E. BEITLICH

Tests to destruction of a reinforced concrete floor. L. VANDEPERRE. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Tests described had two objects: (1) determination

of factors of safety—with regard to initial cracking of concrete—exceeding limits of elasticity of steel and final rupture of a complete structure constructed of reinforced concrete. Calculated on bases of hypotheses commonly accepted in resistance of materials and stability of constructions. This complete structure was a normal floor of slabs and lower ribs of 20 ft. and 25 ft. made in the yard, without particular care or special supervision, by workmen usually employed on similar work. 2. Determination of same dimension comparable with first for calculated permissible loads, and made under same conditions but of which organic conception is based on use of concrete in very thin curved slabs stiffened by their curvature. Tests consisted of loading to breaking point two floors and measuring deflection of principal beams. Preliminary calculations, description of test loads and measurement of deflections are reviewed with description of tests with photographs and diagrams of cracks and of rupture. Tables and diagrams review results of tests and other tables review comparative conclusion as to resistance and cost of two systems tested. In conclusion the following points are cited: (1) Regularity of mixing without particular care. Special supervision has been within about 15 per cent. (2) The factor of safety against initial cracking in normal floor has been less than 1.0 for one of the beams. A mean value would be about 1.2. But unfavorable circumstance as concreting in very dry and warm weather may have made it fall below 1.0. For the special slab it was equal to about 1.17. (3) The factor of safety to exceeding the limits of elasticity of the steel was 2.4 for the normal floor and 3.0 for the special floor. (4) Factor of safety against rupture was greater than 3.6 for normal floor which could not be loaded to destruction but which would have certainly failed by drawing of the steel. It was equal to 3.49 for special floor which failed prematurely by crushing of concrete in compressed portion of marginal beams, a central drum doubly reinforced would have rendered the section of rupture rigid transversely and thus increased security of the whole. (5) All failures by bending by crushing of concrete have occurred by stepping along planes oblique to compressive stress. (6) Refacings or renderings have behaved perfectly up to point of rupture, these inevitable repairs serve then in practice to perfect work from point of view both of resistance and appearance, and it is therefore necessary to be doubly careful in this respect and not to treat them as mere added veneer.—EDITIONS LA TECHNIQUE DES TRAVAUX

Reinforced concrete in Great Britain. A. KIRKWOOD-DODDS. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, 1930.—First reinforced concrete structure designed by Hennebique was erected in Belgium nearly 50 years ago and some noteworthy examples of his early work are to be found in the city of Liege today. Having been convinced by practical experience of possibilities of concrete in combination with steel, Hennebique formulated a complete method of design and in 1892 established his well-known headquarters in Paris, whence the Hennebique system was spread throughout Europe and to almost every part of the world. Great Britain was not far behind France and Belgium in taking up the new method of construction, for in 1896, the late L. G. Mouchel became the pioneer of this form of construction in that country. Much effort was necessary before reinforced concrete was accepted by British engineers and architects, but in 1898 its claims were recognized by the chief engineer to the Mersey Docks and Harbor Board and by the Great Western Railway engineers, and a little later, Mr. Mouchel was appointed consulting engineer for New General Post Office, London. At time of Mr. Mouchel's death in 1909, some 800 reinforced concrete structures had been completed in the United Kingdom. In referring briefly to methods of calculation, it is pointed out that several structures described were designed by means of formulae evolved by Hennebique some 40 years ago, and ever since then used with consistently satisfactory results, a record attributed to the fact that the formulae and the essential factors used in their practical application were derived from scientific tests and actual

experience. Some typical examples of reinforced concrete building construction on a large scale in Great Britain, referred to are: General Post Office Buildings, London.—Comprising two blocks connected by a two-story basement, these buildings have a total volume of 8,500,000 cubic feet. They were designed on the monolithic skeleton principle, all loads, including that of architectural masonry, being carried by reinforced concrete construction. Buildings embody interesting structural details and use of reinforced concrete resulted in a very large reduction of capital expenditure. Royal Liver Building.—This conspicuous landmark on quayside in Liverpool rises 360 ft. above foundation, and is 301 ft. by 177 ft. in plan. Masonry enclosing walls are carried entirely by a monolithic reinforced concrete skeleton. Long span beams, interior arches and main columns, carrying loads up to 1,500 tons each are some features of building. Government Stationery Office, London.—An impressive example of reinforced concrete work, including two eight-story buildings, one 106 ft. by 80 ft. and other 480 ft. by 215 ft., connected by a three-story bridge. Wall panels of skeletons are mainly occupied by windows, providing admission of natural light. Railway Station and Warehouse, Newcastle-on-Tyne.—Building, 430 ft. by 178 ft. in plan by 83 ft. high, comprising high and low level railway stations, with six lines of rails, above them being a three-story warehouse and a flour storage of 500,000 cu. ft. capacity with sloping floors supported by reinforced concrete lattice girders carrying loads up to 300 tons each. After over 25 years of heavy continuous duty, structure still justifies Hennebique methods employed in its design. Dock Warehouses and Grain Silos.—The first example is a range of five transit sheds at Manchester docks, with frontage of 2250 ft. and depth of 112 ft. from front to back, total floor and roof area being nearly 22 acres. Unusual feature is railway tunnel passing obliquely through one of buildings. Among other grain silo buildings in Manchester docks is one with a storage capacity of 1,680,000 bu., the largest structure of its class in Great Britain. Another interesting example is group of six cylindrical silos near Newcastle-on-Tyne, each compartment being 72 ft. high by 45 ft. in diameter, embodied in a structure 176 ft. by 92 ft., 109 ft. high.—EDITIONS LA TECHNIQUE DES TRAVAUX

BRIDGES

Pyke's Creek bridge. GEOFFREY OWEN THOMAS. *J. Inst. Eng.* (Australia), Oct., 1930, V. 2, No. 10, p. 369.—Paper deals with choice, design and construction, and testing of a 5-span continuous reinforced concrete road bridge, 246 ft. long, over a reservoir near Bacchus Marsh, Victoria. With such long spans, considerable economy is obtained by using beams of widely varying moment of inertia, the deep haunches greatly reducing the positive moment in mid-span. Temperature and secondary stresses are investigated. Erection was simplified by incorporating, as reinforcement, in the superstructure, a light welded steel truss, from which forms were supported. Deflection tests were found to agree with calculated values if E is taken as 6,000,000 lb. per sq. in. The average compressive strength of test cylinders was 3770 lb. per sq. in. Tests proved the waste in adhering to value of E equal to 2,000,000 in cases where concrete is properly supervised in construction. Detailed calculations are included.—JOHN E. ADAMS

Working stress of 1,200 lb. used for 39 ft. concrete slabs. CLARK H. ELDRIDGE. *Eng. News Record*, Dec. 18, 1930, V. 105, No. 25, p. 981-982.—Reinforced-concrete slab construction on 39-ft. spans with concrete working stresses of 1,200 lb. per sq. in. was required by the limited head clearance at the Second Avenue extension viaduct in Seattle. Construction includes use of several unusual details, including an unsupported expansion joint through a 15-in. slab, using steel dowels and a novel method of supporting slab on lower flange of a plate girder. Live load was taken as two 20-ton trucks with allowance of 100 per cent on rear axle of one truck. Working stress in steel was taken at

16,000 lb. per sq. in. Under these conditions slab thicknesses for four spans were 12, 13, 15 and 12 in. respectively. Main reinforcement consisted of 1⅛-in. round bars at 4-in. centers for the 36 and 39-ft. spans, with ½-in. round spacer bars at 4 ft. intervals. Concrete was designed for 3,000 lb. per sq. in. and a fiber stress of 0.4 of this value or 1,200 lb. was adopted. Strength of concrete in structure, as shown by 27 test cylinders broken at 28 days, averaged 3,559 lb. per sq. in. with a high strength of 4,760 lb. and a low strength of 2,380 lb. Two expansion joints were provided about 60 ft. apart at right angles to supporting bents. These joints necessitated some method of distributing heavy wheel loads between adjacent slabs. This was done by encasing the edges of the slabs in steel channel sections, anchored into edge of concrete by means of steel straps, and using heavy steel dowels to transmit the load across the joint. Backs of channels are spaced 2½ in. apart to allow for expansion.—D. E. LARSON

Design of arched bridges with fixed supports. H. CARPENTER. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 11-12, p. 612-622, p. 673-685.—Method is given of obtaining moments and stresses due to live load, and is based upon assumption that uniformly distributed live load can be used in place of usual procedure of using influence line. Investigations were restricted to critical sections at crown and springing. From study of properties of influence line maximum positive moment at crown is obtained by loading central part of the arch. Values of moments, vertical reactions at supports and horizontal thrust are obtained using influence line by superimposing unit loads at most unfavorable positions. Maximum values of moments, vertical reactions and horizontal thrusts are thus obtained. However, moment = αwl^2, horizontal thrust = $\beta wl^2/r$ and vertical reaction = γwl, where w = the unit load per ft. and α, β and γ are coefficients. By equating the results obtained by use of the influence lines to these expressions corresponding values of coefficients, α, β and γ are obtained. Values of these coefficients are given for different rise-span ratios and for different ratios of depth at crown to depth at springing. Tables of coefficients are given for parabolic, open spandrel and filled spandrel arches. It is necessary to know values of uniform load w in order to use above coefficients to obtain moments, thrusts and reactions. Values of equivalent uniform load w were computed by use of influence line and actual wheel loads. Equating values thus found to values in terms of w, values of w were obtained. Results of these investigations are plotted giving values of equivalent uniform load w for spans varying from 25 to 400 ft. Curves are plotted for negative and positive moments at springing, positive moment at crown, horizontal thrust at crown and springing, and vertical reaction at springing. Study was made to determine the relation between spacing of longitudinal beams and resultant load imposed upon arch ring. A point of interest from a study of tables of coefficients for moments is that as the ratio of the springing thickness increases, arch axis remaining the same, moments at springing increase, but moment at the crown decreases. The result is a reduction of moment at center and a corresponding increase of moment at support. Application is made of the above method to an open spandrel arch of 150 ft. span, rise of 22 ft. 6 in., depth at crown of 2 ft. 3 in. and depth at springing of 3 ft. 4½ in. Stresses are computed and a comparison made between the approximate method presented and the "exact" method showing close agreement. Value of the horizontal thrust by the "exact" method is 78,000 lb. and by above approximate method is 82,200 lbs.—JOSEPH MARIN

DAMS

New type of arched dam. A. PENA BOEUF. *Summary Reports*, First International Congress for Concrete and Reinforced, Concrete Liege, 1930.—Serious difficulty in calculation of arched dams, in which certain hypotheses must be made to consider relative influence of jutting in the foundation and

the side walls, is radically corrected in type of dam proposed, and in which in addition to suppression of indeterminate factor of elasticity, there is an enormous saving in amount of materials used. Fundamental principle is the division into horizontal isostatic arches and the ease with which the calculation of elasticity can be applied to them.—EDITIONS LA TECHNIQUE DES TRAVAUX

Stresses in dams. S. CAROTHERS. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The usual graphic method of obtaining stresses in dams necessitates beginning at the top and working downwards towards the base. It will thus be seen that any error in earlier part of work will be perpetuated to the end and will vitiate results obtained for lower part of dam. Paper gives a method of obtaining stresses at base by a direct process of calculation and this should prove useful as a check on stresses as found by graphical methods. In addition, the paper introduces polar coordinates which have great advantage that for dams of approximately triangular section the principal stresses are obtained direct for the front and flank of the dam. These are points where stresses have maximum values. Stresses are obtained from Airy's stress function x by methods fully explained in Love's Elasticity.—EDITIONS LA TECHNIQUE DES TRAVAUX

MISCELLANEOUS

Calculation of secondary effects in composite girders. A. PENA BOEUF. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Considering these girders, for purpose of calculation, as an articulate isostatic system, to each diagonal one applies the corresponding tensile stress and when these diagonals are virtually suppressed, the girder is calculated as a Vierendeel girder. To the law of moments resulting at each joint, the distribution is applied to the parts which meet thereon, in accordance with the relative flexibility of each.—EDITIONS LA TECHNIQUE DES TRAVAUX

Reports on new experiments concerning reinforcing of concrete beams against thrust stresses. OTTO GRAF. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 137-42.—Results of three series of tests on reinforced concrete beams are reported; special consideration was given to reinforcing against thrust stresses. Several weak points in German specifications for reinforced concrete of 1925 are discussed and certain changes are proposed.—A. E. BEITLICH

Tests on columns of bound concrete reinforced with bars of special steel and welded. R. SALIGER. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—1. Tests deal with columns of bound concrete with a high percentage of reinforcement of special steel. 2. Load carried results in every case from the sum of the resistance to compression of concrete core and longitudinal reinforcement as well as from resistance to extension of binding. 3. The share of resistance of concrete amounts to $\frac{1}{3}$ to $\frac{1}{6}$ according to importance of reinforcement, which reinforcement supports from $\frac{2}{3}$ to $\frac{5}{6}$ of the total load. 4. Welded longitudinal reinforcement has a resistance to compression 10 per cent inferior to that of non-welded bars. 5. On account of their high resistance to compression the section of such columns should not occupy more space in the structure than columns of rivetted steel.—EDITIONS LA TECHNIQUE DES TRAVAUX

Design of circular tank of reinforced concrete. I. L. RASMUSSEN. *Eng. News Record*, Dec. 11, 1930, V. 105, No. 24, p. 923-925.—The designer of circular tanks of reinforced concrete has to overcome theoretical obstacles not generally met with in design of other structures. The walls of the tank are

rigidly fixed to the base, which, for purpose of analysis, may be considered immobile. The radii of the cylindrical sides tend to lengthen under pressure. At base, the walls are completely restrained and their radii cannot change in length. As distance from base to any horizontal section of wall increases, the unrestrained movement of walls also increases until a point is reached where movement is entirely unrestrained. To determine this distance and the moments and stresses which are set up between the base and this point through the restraint of the base is chief problem to be solved in design of circular tanks. The author presents a simple and direct mathematical analysis for determining these moments and stresses.—D. E. LARSON

Articulated concrete revetment construction on the Mississippi River. MORRIS W. GILLAND. *Eng. News Record*, Dec. 25, 1930, V. 105, No. 26, p. 996-1003.—Early in the task of regulating the Mississippi River for navigation and flood protection, it was manifest that some of its banks must be prevented from caving. Gradually brush mattress superseded all other types of revetment structure until 1915 when its position began to be contested by concrete. Concrete was the first thought because it had served well for upper bank paving, was even more durable under water and suffered little from being alternately wet and dry. The articulated-mat revetment being used at present consists of a series of lapped mats extending from waters edge to bottom of underwater bank and outshore on river bottom some 50 ft. farther. Each mat is 140 ft. wide alongshore and from 300 to 350 ft. long outshore from the water's edge. Adjacent mats overlap 10 ft. Each mat is made up of sections, 4 by 25 ft. strung on steel cables, which extend upbank and are anchored inshore. Construction consists of four operations: (1) fabrication of articulated mat sections; (2) grading upper bank to 1 on 4 slope; (3) assembling and sinking sections to make continuous revetment; and (4) paving sloped bank with concrete. Each of four operations requires separate plant (c. f. A. C. I. JOURNAL, June, 1930, p. 799, *Proceedings* V. 26, 1930).—D. E. LARSON

Nomogram for calculating average diagonal tension in a given section of beam. JORGE QUIJANO. *Ingenieria* (Mexico), Nov., 1930, V. 4, No. 11, p. 422 and 427. A simple nomographic chart, one of a series for use in reinforced concrete design, for investigating diagonal tension in beams. Left hand scale, diagonal tension T; center scale, difference in moment between sections A and B; and right hand scale, depth d for various angles of stirrup. All in metric units, with explanation of use on chart and opposite page.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating simple rectangular slabs and beams. JORGE QUIJANO. *Ingenieria* (Mexico), Oct., 1930, V. 4, No. 10, p. 390-391.—A simple alignment chart, one of a series for use in reinforced concrete design. Left hand scale, depth of slab d, middle scales, moment M and area of steel A^s, and right hand scale width b, all in metric units. Explanation of use is given on the chart and opposite page.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating slabs freely supported or partially restrained. JORGE QUIJANO. *Ingenieria* (Mexico), Oct., 1930, V. 4, No. 10, p. 378-379.—A simple nomographic chart, one of a series, for use in reinforced concrete design to determine thickness of slab and area of steel. Left hand scale, span; center scale, depth of slab and area of steel; and right hand scale, load; all in metric units. This chart is for f_s = 20,000 lb. per sq. in., f_c = 900 lb. per sq. in. and n = 15. Explanation of use is given on chart and opposite page.—C. G. CLAIR AND M. N. CLAIR

Reinforced concrete bunkers in Germany. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 669-673.—During recent years large number of coal and coke bunkers have been constructed in Germany, some notable for long spans and heavy loads on main girders, and provision made

for possible settlement of ground due to mining operations. Total height of reinforced concrete bunker at Alsdorf is 170 ft. The capacity is 3,000 tons of coal. It is 78 ft. high and 79 ft. by 49 ft. 3 in. in plan. Due to the large space required by the cable drums, it was necessary to cantilever upper part of the structure on two sides. Foundation slab is 62 ft. 3 in. by 49 ft. 3 in. by 4 ft. thick with ribs projecting to a height of 11 ft. 6 in., thus reducing the maximum pressure on the ground due to dead and live loads and wind, to 3¾ tons per sq. ft. Owing to possibility of subsidence due to mining operations, the foundation was designed to resist stresses arising from uneven settlement of ground, and it is possible for whole structure to be inclined at angle of 5 degrees with vertical before dangerous stresses occur. Bunker is supported on four main columns, 16 ft. 5 in. by 4 ft. 1 in. cross section. Provision has been made for restoring structure to its original position if any settlement or canting takes place. Structural details of bunker are given.—JOSEPH MARIN

Angular rotation method for the analysis of multiple span frames. R. A. FREEMAN AND A. S. GILMAN. *J. Boston Society C. E.*, Dec., 1930, p. 543-562.—There is given the development of the fundamental equations for analysis of statically indeterminate frames based on the work of Ernest Suter entitled, "Die Methode der Festpunkte," published in 1923. Two numerical examples are worked out to illustrate the method. The angular rotation method is particularly valuable for analysis of multiple-span rigid frames.—MILES N. CLAIR

Thin reinforced concrete slabs. J. H. H. WILKES. *Roads and Rd. Constr.* (England), Dec., 1930, V. 8, No. 96, p. 434.—In case where steel is cheap, it may be more economical to employ floor slabs in bridges, or similar structures, in which the steel is greater than normal. In this case strength of slab will be determined by that of concrete. Curves show relationship between moment of resistance and percentage of steel, for various stresses in the concrete or the steel. Shear will be limiting factor for thickness.—JOHN E. ADAMS

Field inspection of concrete pipe culverts. R. W. CRUM. Iowa State College *Eng. Expt. Station* Bull. No. 99, Sept. 3, 1930.—In 1915, a question having arisen as to suitability of concrete pipe for use as highway culverts, Iowa Highway Commission and Engineering Experiment Station of Iowa State College cooperated in making a field study of such structures. Forty-six culverts ranging in size from 24 to 66 in. diameter were inspected. Inspections showed older designs to be inadequate, and since 1915 stronger and heavier pipe have been used. Records are given of certain culverts covering a life of 22 years, as oldest pipe examined in 1915 was then about 9 years old. Method of inspection was to crawl through pipe, making a thorough examination of all significant features by means of a flashlight. Principal opinions are: Concrete pipe made of good concrete and used in locations in which load on pipe does not exceed its supporting strength, and so installed that water flowing through (hydraulic traffic) cannot undermine support for the pipe, will make satisfactory culverts that can be classed as permanent as other concrete construction. The culverts examined did not exhibit progressive deterioration to an extent great enough, or in such a way, that predictions as to the length of life could be made therefrom. Culverts were found to be retired from service for two reasons—cracking, due to overload, and failure of lower end, due to undercutting by stream. Latter failure can be prevented by proper design and installation. Results of detailed examination of individual culverts are given. During the life of culverts covered by these inspections, pipe were installed without much reference to relation between strength of pipe and height of fill for particular locations, but investigation in 1915 showed that older pipe were not strong enough and at various later periods stronger designs were used. Necessity for standardization of materials and manufacturing

conditions as well as of design, is indicated by the fact that two plants working under the same design produced pipe of considerably different strength. Greatest destructive force acting upon concrete pipe culverts was pressure of fill supported by pipe including effects of truck wheels and other super-loads. Three defects in installation which affect the serviceability and durability of the culverts were: (1) open joints between sections; (2) culverts laid on too flat a grade line so that pipes became more or less filled with mud; and (3) culverts installed with outlets subject to undermining by cutting back of stream channel. The latter two defects might occur with any type of culvert. A point not to be overlooked is that although certain culverts may have various defects, they may still be performing their major function. Marston's method of computing probable imposed loads on culverts is included, together with curves for facilitating such computations.—R. B. B. MOORMAN

Foundation problems and their solution. L. R. VITREBO. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 301-303.—A footing is a structural unit used to distribute wall or column loads to foundation materials. A spread footing distributes load directly to soil without use of any other intermediate structural members. Such footings are nearly always made of reinforced concrete to save in excavating, materials and weight of footing proper. Five classes of spread footings are: (1) wall footings, (2) independent column footings, (3) combined footings carrying more than one column, (4) cantilever footings and (5) raft or mat foundation. In ordinary type of building, that footing should be chosen in which live load bears highest percentage to dead load, and its area should be determined for total load at allowable soil pressure. Pressure of dead load per unit of area should be determined, and area of all other footings should be proportioned for dead load only, with this unit pressure. Independent spread footings are usually square or rectangular for practical reasons. Those with sloping tops contain less concrete than stepped footings, but. unless slope is flatter than 2 horizontal to 1 vertical, the top will require forms which offset any saving in concrete. Combined footings are simply inverted girders, or slabs receiving an upward pressure equal to carrying capacity of soil, throwing their end reactions to the columns which bear upon them. Cantilever footings are often resorted to when combined footings are expensive or impossible to use, that is, when it is necessary to make use of land close to property line and still not encroach upon adjoining property. Raft or mat foundations are used for soils of low bearing capacity. The foundation is then spread over whole area of building using either flat slab or beam and slab construction.—D. E. LARSON

Reinforced concrete and its hypotheses. FERNAND DUMAS. *Génie Civil*, Dec. 6 and 13, 1930, V. 97, No. 23-24, p. 561-564 and 589-593.—Reinforced concrete theory is based on a series of paradoxical results of inexact and incomplete experiments. Various official circulars make no effort to conceal the crude approximations. Research has concentrated on elaboration of detail. Yet how much more important for the design of a section is exact determination of coefficient $n = 10$ or 15, than slight variations in allowable stresses? Design, instead of seeking to establish actual stresses produced by forces acting on a certain member, attempts only "to prove sufficient safety of structure," according to the German authority, Mörsch. The weakness of this procedure is admitted in 1906 specifications, requiring that tensile strength of concrete be taken into account under certain conditions. But what to do in the case of hyperstatic structures? A study of deformation of reinforced concrete structural members is greatly neglected. Deformation measurements with Manet-Rabut apparatus are accurate only in rare cases of simple tension or simple compression. Deflection measurements of flexural members yield results widely diverging from computed design values. The author attempts to throw light on the actual agreement of fundamental hypotheses of design with the basic properties of reinforced concrete and to test application of

elastic theory and theory of strength of materials in this case. Reinforced concrete is a heterogeneous material in its constitution and its fabrication. Its three constituents are: steel, concrete in tension and concrete in compression. Of these steel is least variable and should be—yet is not—standard of comparison. Early experiments by Considére and Bach show that properties of concrete are greatly altered in presence of reinforcement. Is it correct to apply theory of elasticity to this heterogeneous combination? Does law of continuity hold in this case? It seems, that elastic theory furnishes us only with approximate indications, which are to be treated only as average values. Heterogeneity due to fabrication upsets elemental theory of strength of materials, whose laws deal with simple tension, compression, flexure and torsion, on condition that shear is equal to zero. The assumption is that *some transverse section remains plane*, identical with its original position and perpendicular to neutral axis during deformation. The author takes up in succession compression, tension, flexure, transverse deformation and torsion of reinforced concrete and comes to the conclusion that the above assumption is inexact for reinforced concrete and that results of this hypothesis necessarily disagree with actual conditions.—M. A. CORBIN

Formulae and curves for the calculation of members in reinforced concrete in bending. F. CAMPUS.—*Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The formulae proposed permit making all bending calculations for ordinary members in reinforced concrete of rectangular or T section singly or doubly reinforced bending being eventually accompanied by compression or tension. In all these cases calculations are carried out by analogous formulae and the same coefficients of which values are given by five graphs. The simple manner in which the coefficients occur in the formulae shows clearly influence of various factors. These formulae can receive terms of adjustment to allow for compression in the ribs of T sections, and equally can be simplified to allow of rapid calculation. The formulae and curves are suitable for all values of coefficient of resistance and of elasticity of concrete and steel.—EDITIONS LA TECHNIQUE DES TRAVAUX

Calculation of rigid structures by the method of assumed static moments. J. RIEGER. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Method presented is called method of assumed static moments which consists of a geometric interpretation of equations provided by the theorem of Castigliano. A construction of several spans being given, it is always possible to divide it by the spans into several intermediate constructions for which conditions of deformation can be written as functions of assumed static moments with moment diagrams in relation to certain axes. These equations combined with static equations given the solution of the problem which is expressed in the general formulae by the form $M_m = \alpha_m A\ \beta_m B + \gamma_m C + \ldots \chi_m U + \varphi_m V + \xi_m Z$ in which the symbols indicate: M_m, moment in some joint of the construction given; $A, B, C, \ldots U, Y, Z$, certain functions of loads applied to the construction. $\alpha_m \beta_m \gamma_m \ldots \chi_m \varphi_m \xi_m$ being functions of the form of the construction studied. It is always easy to tabulate the functions $A, B, C, \ldots U, Y, Z$, and the coefficients $\alpha_m \beta_m \gamma_m \ldots \chi_m \varphi_m \xi_m$ to facilitate the use of the method.—EDITION LA TECHNIQUE DES TRAVAUX

Reinforcement of panels and slabs. H. LEITZ. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The progress of the use of panels and slabs as elements carrying load in reinforced concrete structures necessitates extension of theory of reinforced concrete to calculation of reinforced walls subjected to forces acting in their own plane, combined with the study of reinforcement of slabs. Under heading 1 (uncracked concrete) the usual hypotheses can be used to calculate the tensions

in the concrete and steel as well as for the arrangement of the reinforcement. The case of pure shear always occasions difficulty notably when reinforcements have finite thickness. The calculations for the reinforcement of a wall must be based on the hypothesis of cracked concrete (heading 2). The simplest and most certain reinforcement comprises two families of reinforcements following the lines of equal stress which at once raises difficulties on account of the curvature of these lines. For this reason two parallel sheets of straight reinforcement are very often used, and applied in various ways. The principal difficulties arise with the shear stresses. We have to admit that the concrete may act as a compressed diagonal in a direction inclined at 45 degrees to the reinforcement but we must expect large deformations following lines of principal tension. In case of heavy shearing loads and variable applications, three layers of reinforcement are necessary. Just as a deflected sheet may be considered as a pile of thin sheets, the same consideration may be applied to the corners of slabs simply supported.—EDITIONS LA TECHNIQUE DES TRAVAUX

ROADS AND PAVEMENTS

Concrete pavements. D. J. GARLAND, *J. Inst. Eng.* (Australia), Oct., 1930, V. 2, No. 10, p. 396.—The various well known methods of estimating proper thickness of concrete roads are reviewed, including Goldbeck's simple formula $h = \sqrt{\frac{3P}{f}}$ where h = depth of slab in inches, P = load in lb. and f = safe tensile strength of concrete in lb. per sq. in., also Prof. Westergaard's more elaborate treatment. In actual practice, a standard thickness is usually decided upon and foundations prepared where necessary. Since engineers have better knowledge of thickness of macadam required for various conditions, experimental results of comparisons between the thickness of macadam required and of an equivalent concrete slab are discussed. The results state that (a) for a given load intensity of pressure under a macadam road is 8 times pressure under concrete slab of same thickness, and (b) area of distribution of pressure under a concrete road is at least four times as great as under a macadam road of same thickness. These areas are actual areas over which pressure is recorded and not equivalent or effective areas. Result (a) appears more reliable. Accepting this data and assuming conical distribution of pressure to the foundations it is shown that if h' = macadam thickness and h'' = thickness of equivalent concrete slab, $h' = \sqrt{8h''}$. For greater safety the factor may be taken as less, say, 4, when $h' = 2h''$. In an example it is proposed to substitute a 5-in. concrete slab and prepared sub-grade for a 12-in. macadam surface. If $h' = 2h''$ equi valent macadam thickness for concrete slab = 2x5 = 10 in. and thickness of sub-grade required = 12 — 10 = 2 in.—JOHN E. ADAMS

ARCHITECTURAL DESIGN

The architecture of concrete. BERESFORD PITE. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Consideration of effect of primary distinction in method between an architecture of stone or brickwork and an architecture of concrete is motive of this paper. The idea of internal adhesion belonging to material producing homogeneity without anxious care must be novel, and of nature of a relief to the constructor; its attainment has always been his ideal, but the repugnance to the employment of concealed tierods for cohesion in walling is always manifested by the consistent tendency to rely either on trabeated or arcuated lines. The new idea, as our brethren on the Continent have perceived, is revolutionary rather than evolutionary. It raises manifold questionings as to

due architectural expression, and necessarily also it involves reconsideration of methods in study of practice of design, with courses of training leading to its fulfillment. It would appear to be consequential, if we had not experience of antiquity to the contrary, that a fundamental constructive method must deflect architectural expression of construction from traditional motives. Such expression is at least one of the primary causes of any architecture, though its caricature as decoration may survive and invade other materials and methods. Today we are in the presence of a new process that is rapidly becoming general and therefore little apology is required for an attempt to discuss its manifestation in architecture and its effect upon our ideals. The subject is sketched under three heads: 1. The difference in constructive method and its expression in design. 2. The resulting architectural forms and the possibility and service of ornament. 3. The problem of the aesthetic use of concrete architecture.—EDITIONS LA TECHNIQUE DES TRAVAUX

Architecture and reinforced concrete. E. BALIS. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—So weak is our poor spirit—said Suger—that it attains to truth only through the medium of realities perceptible to the senses. Truth is the spirit and the chief element of architecture, realities being the forms of the art. We may distinguish true and false forms. An unfortunate tendency of our times is to consider details while slighting the generality. This is a point brought out indicating our conception of new architecture and the forms proper of reinforced concrete. Architecture is an art and therefore something more than a technique.—EDITIONS LA TECHNIQUE DES TRAVAUX

The architecture of concrete and reinforced concrete. A. POMPE. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The aesthetics—still undetermined—of concrete and reinforced concrete are subject, like those of wood, brick and stone, to certain laws. These laws are themselves subject to the spirit of the time; they evolve in accordance with those material and immaterial conditions that govern the "directive" periods which follow on one another through history. That is to say, a "style" is far more an emanation from the spiritual and sentimental character peculiar to a given civilization, than it is the necessary and inescapable expression of the proper handling of a given material, the use of which happens to have been preponderant during that civilization. Style is an intermittent and periodic phenomenon, whereas the art of building is a permanent fact, though its value may vary. Style is expression: architecture is but a mask applied to building with more or less of expressiveness and truth. Wood, brick, stone, steel and reinforced concrete, each has its proper technique; that of rolled steel is a modern thing, and that of reinforced concrete is more recent still. In architecture steel has never properly succeeded, for a number of reasons. Reinforced concrete has only now entered upon a really active phase, and there is reason to believe, for other reasons, that this material will have an influence over the architecture of tomorrow much greater than that of its elder brother steel.—EDITIONS LA TECHNIQUE DES TRAVAUX

Four modes of construction in a single settlement or subdivision. *Betonwerk* (Germany), Dec. 7, 1930, V. 18, No. 49, p. 688.—Toward the end of year 1928 the Leipzig Fair Association presented a plan to the Sachsischen Arbeits and Wohlfahrtsministrium, to build four apartment buildings of 24 families each of steel, concrete, wood and brick respectively. The idea is to afford an opportunity to study each type of construction in a comparable manner. It is hoped that economical and technical phases of each class of construction may by this method be better observed. Each building is about 200 ft. long by 21 ft. wide and 3 stories high. The whole idea presents a building material exhibit in which many of building specialties can be seen. Each building is illustrated and described in detail. The entire enterprise is nearing

completion and is under direction of Rudolph Stegeman, Government Building Advisor, Leipsig. It is an experiment which up to this time has not been tried, and in due time the Leipsig Builders Fair will issue a complete report.—H. FRAUENFELDER

FIELD CONSTRUCTION

BRIDGES

Precast light weight concrete slabs solve bridge floor problem. *Concrete*, Jan., 1930, V. 37, No. 6, p. 15-16.—New type of bridge floor, consisting of large precast slabs of light-weight concrete welded to steel floor system, has been developed by Division of Bridges of City of Chicago. The Wabash Ave. bridge has slabs over the entire moveable part of the bridge floor, including roadway. Twenty-eight of largest size slabs, 102 sq. ft. in area, were required, twenty-eight 73 sq. ft. in area, and 56 smaller ones between street car tracks and rails. Thirty sidewalk slabs 98 sq. ft. in area and 2½ in. thick, were required. Roadway slabs have total thickness of 5¾ in. and are reinforced with steel bars in top and bottom, and stirrups. Sidewalk slabs, of light-weight concrete, are 2½ in. thick, and reinforced with layers of welded wire fabric, held 1 in. apart. Cement and aggregates employed were 1:1½:2½ mixture of Incor cement, No. 3 torpedo sand, and coarse haydite, with 4½ gal. water per sack of cement. Slabs were cast by aid of electric vibrating process that vibrated steel reinforcement. Molds were in horizontal position. Reinforcement was anchored firmly. Four holes were provided in each slab into which 1-in. pipe sleeves were fitted to facilitate handling, and later filled with mortar. Each slab was also anchored by U-bolts passing under stringers and up through holes in slabs where loose plates were held with nuts. Anchoring device removed after slabs were welded into place. Slabs were lifted into position by steam derrick equipped with two sets of cable spreaders fitted with hooks on each end. Final operation was welding of slabs to steel floor system, by means of ⅜-in. protruding steel plates.—C. BACHMANN

Winter concreting on a three arch bridge. RICHARD F. REY. *Eng. News Record*, Dec. 25, 1930, V. 105, No. 26, p. 1004-1005.—Continuous prosecution of concrete work on main-arch spans in severe winter weather and a nice procedure in handling expansion joints were employed in construction of Logan St. bridge in Lansing, Mich. The Logan St. Viaduct is 1,075 ft. long and has three river spans 61, 67, and 90 ft. long, 534-ft. of reinforced concrete viaduct, and 126-ft. of concrete-incased steel beam viaduct and earthfill approaches between retaining walls. Portions of bridge constructed in winter were housed in a framework covered outside with ship-lap siding sealed inside with building paper and roofed with tar paper. The housing provided ample space for building forms and handling concrete. Unit steam heaters kept enclosure warm. These were standard heater units such as are used for heating industrial and commercial buildings, instead of pipe coils or cast iron radiators. Outside temperatures varied from around freezing to 10° below zero, and inside temperatures were kept between 60 and 70° for first part of heating period and from 50 to 60° for last part. Concrete was made from aggregates heated by steam-pipe grids underneath stock piles and water heated by passing through old steam boiler. Temperature of concrete entering forms was between 60 and 80°. Vertical 1-in. expansion joints were carried across bridge over river piers and at 100-ft. intervals throughout viaduct portions. Butt joints were employed. Double columns and column girders were constructed throughout to eliminate any sliding expansion joints.—D. E. LARSON

BUILDINGS

The demolition of several reinforced concrete buildings at Brunn. R. BORTSCH. *Summary Reports*, First International Congress for Concrete

and Reinforced Concrete, Liege, 1930.—The machine factory of Brunn formerly Brand and Lhuillier disused for several years, was bought by the municipality which undertook in 1927 demolition of numerous buildings. By reason of existence of dwelling houses in immediate neighborhood, destruction of several halls and storied buildings in reinforced concrete of comparatively recent construction had to be undertaken with great care. The slabs were destroyed with a hammer. Ends of girders were cut with a pneumatic drill and reinforcement divided by an oxy-hydrogen flame. Frames of halls 13 meters high were overthrown in turn and cut up on the ground. Bases of columns were cut, then frames were overturned by hauling on cables with a windlass. Division of columns and girders of large dimensions was carried out after overturning by means of dynamite. For reasons of safety pieces to be destroyed were covered with hurdles held between metallic trellis work. Certain parts of buildings with high stories of 55 ft. were destroyed in one piece by dynamiting columns. Three months were taken in demolishing reinforced concrete buildings containing 12,000 cu. yds. Cost was 127,000 Belgian francs.—EDITIONS LA TECHNIQUE DES TRAVAUX

MISCELLANEOUS

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Jan., 1931, V. 38, No. 1, p. 35-40.—Second half of article on estimating labor and material quantities gives figures for definite areas for all units of form work. Hours of labor required are expressed as equivalent number of hours for one man. Lumber, other materials, and labor are each detailed. Time when each type of form may be removed is given.—C. BACHMANN

Concrete calendar. "Beton Kalender 1931." 1930, Wilhelm Ernst and Son, Berlin, 2 parts, 25th year, 1020 illustrations, R. M. 7.50.—Reviewed in *Bautechnik* (Germany), Sept. 19, 1930, V. 8, No. 41, p. 629.—This handbook for concrete and reinforced concrete construction is one of most common books and reference work for German contractors and concrete investigators. Two volumes are greatly enlarged and revised, especially chapters on slabs, tanks, arch bridges and girder bridges. Chapters on calculation of reinforced concrete cross-sections, solid floor construction and ground construction are new. New specifications for cement, mortars and concrete of several countries are given and great number of general tables are included. A. E. BEITLICH

New improved movable conveyors for cast concrete construction. RIEDIG. *Bautechnik* (Germany), Sept. 5, 1930, V. 8, No. 38, p. 567-8.—Great economy of movable conveyors for structural materials and their great field of application led more and more to their introduction in modern cast concrete construction. Two new patented conveyors are described with 66 and 44 tons per hour capacity. Materials are delivered to conveyor belt in first case by means of two screws and in the other case by means of a scraper. For concreting of quays, long walls or canals a new traveling concrete distributor is described handling 1400 cu. ft. per hour. A 2-yd. concrete mixer is mounted on base of crane. A number of minor improvements on travelers and chutes are illustrated.—A. E. BEITLICH

Crushed-stone plant builds profits by producing pre-mixed concrete. *Pit and Quarry*, Nov. 19, 1930, V. 21, No. 4, p. 56-58.—A new central-mixing plant has been erected in connection with crushing and screening portion of Hawthorne plant of Samuel Bracus Sons, Paterson, N. J. Eighteen reinforced-concrete storage bins of combined capacity of 5,000 tons and stock-piles assure a ready supply of aggregates. Two 3-compartment batcher bins are arranged side-by-side. Batching hopper of one bin can discharge dry mixes into transit-mix trucks or into a 3-cu. yd. stationary mixer, as desired. Mixer discharges

wet mixes into agitator trucks at one side. Other bin is for dry-batching stone, sand and cement, and it discharges into multi-batch trucks to be used in road construction. All proportioning is by weight. Sacked cement is received in truckloads and stored. Sacks are opened by hand and cement dumped through a grizzly in floor to conveyor which runs to a cement hopper at side of one batcher bin whence screw conveyor transfers the cement to weighing hopper. Firm operates ten dump-body and four transit-mix trucks.—A. J. Hoskin

A caisson foundation job. O. E. Mogensen. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 311-314.—The open-well type of caisson foundation, used extensively in the vicinity of St. Louis, is well illustrated by the foundations for the Missouri Portland Cement Company's new early strength Velo plant, built during 1927 and 1928. To ascertain practicability of this open-well method, it was decided to sink a concrete caisson to support the high concrete chimney, lined with fire brick at top. The chimney is 19 ft. in diameter, 220 ft. high, and weighs about 1,100 tons. The caisson which was 19 ft. 6 in. outside diameter, with a wall thickness of 3 ft., struck fire clay 42 ft. below the ground surface. The first section of caisson, cutting edge, was 10 ft. high, and was made from company's high early strength cement. The entire caisson was sunk quickly and without trouble. Other structures for which caisson foundations were used consisted of eight cement silos in clusters with interspace bins, each of the silos 32 ft. in diameter by 70 ft. high; four rock silos 35 ft. in diameter by 60 ft. high; and four clinker silos also 35 ft. in diameter by 60 ft. high. Materials to be stored in these silos weigh from 100 to 115 lb. per cu. ft. The three silo structures weigh, respectively, when filled, 32,620 tons, 16,640 tons, and 18,560 tons.—D. E. Larson

Placing concrete under water. Cayetano Lugo. *Ingenieria* (Mexico), Oct., 1930, V. 4, No. 10, p. 407.—Author contends that usual conception of action of the tube (tremie) in placing concrete under water is erroneous. He suggests careful attention to use of graded aggregates and of a number of tubes rather than one.—C. G. Clair and M. N. Clair

Roads and Pavements

New system of concrete paving. *Rds. and Rd. Constr.* (England), Jan. 1, 1931, V. 9, No. 97, p. 24.—A system of paving is described, in which large, unsightly cracks are replaced by invisible hair cracks in much greater numbers. Instead of every precaution being taken to provide easy sliding of the slab over the foundations as expansion or contraction takes place, an unreinforced wearing surface is suitably anchored to a reinforced under-slab, which is again anchored into the sub-soil. In case of a clay sub-soil, for example, foundation is dusted with fine ashes or sand, and gridded with trenches 4 to 6 in. deep at say, 7 ft. 6 in. centers. A slab of 3 in. or more, thickness is poured and reinforced with at least 0.5 per cent steel. The top surface of this is indented with specially shaped elliptical recesses, and is provided with vertical pins for anchoring the top layer of concrete, and gaging its thickness. These pins also are used for indicating position of indent forms, and for holding reinforcement clear of ground. Top slab is then poured on this sub-base, and through indents and anchors, is made to break up into small squares each separated from one another by minute hair cracks caused by excessive tension. The ribs in the foundation cause a wide distribution of pressures, and continuity is provided. Less care need be taken in curing with this method of construction.—John E. Adams

Shop Manufacture

Manufacturers like winter work. *Concrete Products*, Dec., 1930, V. 39, No. 6, p. 13.—A symposium of data from a large number of subscribers re-

garding the winter manufacture of concrete products shows that many manufacturers favor winter manufacture, at least on a reduced scale, for the purpose of holding their best men. It also allows them to accumulate stock, especially in case of block manufacturers, for an early spring building campaign. It is generally believed that the additional cost of products made in winter is negligible, although there may be some delay in the handling of frozen materials. The opinion is expressed by one burial vault manufacturer that winter is a better season for the burial vault business, and it is his belief that vaults cured in winter are of better quality.—E. S. HANSON

Profitable manufacture of concrete building units. FRED A. SAGER. *Concrete*, Jan., 1931, V. 38, No. 1, p. 17-20.—An illustration of the production ills of a large manufacturing concern are described and the prescribed remedial measures outlined. Interest to products industry in the example is possibility of reducing stocks carried, by smoothing out production curve with fluctuating delivery curve, by a careful control of a few major items of plant's product. As an application of method to the products business, the output for one year, based on the actual experience of a plant, is given in the first four columns of a table reproduced. The second column shows the number of the item in column (1) sold during the year. The third column converts amounts in column two to a common base. Fourth column shows production in "equivalent units" for each item, expressed as percentage of total production, a glance showing that the major production was in five items out of sixteen listed. By including in Group A all items over 5 per cent of total production, and in Group B all items under 5 per cent, we find 90.28 per cent of the production in five items of Group A and 9.79 per cent in 11 items of Group B. By study of production and stock, so far as five items of Group A are concerned, control of the major part of the output is accomplished and items in Group B can be handled by simple method as to stock carried and as a fill-in at convenient times, as to production. Minimum stock is determined as follows: First, the total stock carried shall be equal to the maximum month's sales for preceding year, or estimated maximum month's sales for coming year, if they are expected to vary materially from preceding year, to which is added one-half month's production. Second, stocks for each of major items, Group A, should be equal or nearly equal to maximum month's sales of that item. The stock for the items in Group B should be carried on average about one-half of an economical production quantity.—C. BACHMANN

Hollowspun concrete piles in the making. H. F. HEDDERICH. *Concrete Products*, Nov., 1930, V. 39, No. 5, p. 12-14.—Method followed by Westinghouse Electric & Mfg. Co., in making concrete piles is given in detail. Company has for past six years been producing concrete lighting and trolley posts by the centrifugal method and has recently applied same method to manufacture concrete piles. Piles are round and uniformly tapered for their entire length, increasing ¼ in. in diameter for each foot of length. The ultimate strength of the pile is governed by the compressive strength of the tip. The design is such that failure will occur at this point before the pile is stressed to its limit as a column. All of piles are provided with facilities for jetting. Concrete mix consists of one part cement to three parts of combined aggregate. Water is controlled to give concrete proper consistency for placing and spinning. Material used for both coarse and fine aggregate is very hard, highly dolomitic limestone with cubical fracture. It is very thoroughly washed during its preparation, and is received entirely free of dust. Mold is rotated at 400 r. p. m. for ten minutes. During the process of spinning, the solid particles (the aggregate and cement) are driven outward by centrifugal force and uniformly compacted against the inside surface of the mold and around the reinforcing. This brings all the entrapped air, a large amount of water, and some of the finer aggregate and cement, to center. A hole is formed throughout the length of the column by this action, and as the spinning continues most of the fine solids are packed around periphery of this hole.—E. S. HANSEN

Materials

Admixtures

The chemistry of Rhenish trass. Heinrich Hart. *Tonind. Ztg.* (Germany). Jan. 15, 19, 1931, V. 55, No. 5, 6, pp. 65-6, 84-6. By means of solutions of benzene in bromoform, trass from different places along the Rhine was separated into several fractions. Trass is of variable mineralogical composition and seems to be an alkali aluminosilicate in varying stages of hydrolysis. —F. O. Anderegg

Cement admixtures, their action and absorbing power. P. Mecke. *Tonind. Ztg.* (Germany). Jan. 8, 1931, V. 55, No. 3, p. 33-6.—A few tests, made with rather large amounts of water glass, oleic acid or other water repellent substances, did not yield very encouraging results.—F. O. Anderegg

Aggregates

Specifications for sand, gravel and crushed stone. *Tonind. Ztg.* (Germany), Jan. 5, 1931, V. 55, No. 2, p. 21-2.—Using woven wire sieves up to 1 mm. and round openings for separating larger sizes, separations of aggregates are made at openings: .0024, .0035, .0079, .0236, .0394, .118, .276, 3/8, 5/8, 3/4, 1 1/4, 2, 2 3/8 and 2 3/4 in. For railroad ballast separation is made at 3/8, 3/4, 1 1/4 and 2 3/8 in.—F. O. Anderegg

Tentative standard specifications for gradation of sand, gravel and broken stones. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 37-8.—Report is given of third meeting of committee for standardization of aggregate grain sizes, held on Oct. 31, 1930 in Berlin (Germany). Specifications cover natural sands and gravels, broken rocks, concrete aggregates and materials for railway roadbed construction work. New fixed definitions for certain grain sizes are proposed.—A. E. Beitlich

Thermosit compared with natural pumice. Heinrich Hart. *Betonwerk* (Germany), Aug. 3, 1930, V. 18, No. 31, p. 432.—Chemical analysis, comparative porosity, weight, capillarity and strength of both natural pumice and artificial pumice, "Thermosit" are discussed, and the use of these materials as a light weight aggregate in actual work described. The article is fully illustrated with reproductions of photographs and tables of tests.—Herman Frauenfelder

The application of protective mortar coatings containing metal aggregates. Georges Dupont. *Ciment* (France), Dec., 1930, V. 35, No. 12, p. 462-74.—Concrete exposed to sea water and other conditions where it is subjected to chemical or frost attack or to erosion by flowing water should receive an impermeable and durable protective coating. Metallic aggregates are often suitable for making mortars with cement for this purpose.—F. O. Anderegg

Inert aggregates and the mechanical properties of concrete. R. Dutron. *Rev. des materiaux construction trav. publics*, July to December, 1930, No. 250 to 255.—Article in six instalments emphasizes importance of nature, shape and grading of inert aggregates. Highest compressive strengths are obtained with concretes using river sand and gravel, rounded in shape and well graded. The elastic coefficient being a direct function of compression strength, the same applies in this connection. The reverse order prevails, when results of tensile and flexural strength are considered. The rounded aggregates of river origin yield lowest tensile strengths. Ratio of tensile to compressive strength of concrete varies with age of concrete and with aggre-

gate used. It is highest for crushed aggregate and lowest for river sand and gravel. This ratio varies from 1/10 to 1/20, when compressive strength is tested on 7.9 in. cubes. Shear is a linear function of compressive strength. Its ratio to the latter varies from 2/10 to 3.5/10, when tested on cubes with cross section of 62.0 sq. in. Impact strength is greatly affected by the nature of inert aggregates, the rounded river sand and gravel being most brittle.—M. A. CORBIN

Laboratory studies in the prevention of segregation. A. T. GOLDBECK. *Crushed Stone J.*, Nov., 1930, V. 6, No. 11, p. 3-7.—The theory of segregation is stated as follows: When aggregate, falling in a single stream, piles up in the form of a cone, surface voids entrap pieces of smaller diameter, therefore causing pieces of larger diameter to roll to the outside. Studies were made to develop a method to prevent piling up of material in the form of a cone. A model bin and railroad car were constructed to scale of one inch to the foot and stone screenings approximately the same scale were used for aggregate. Ten possible methods of preventing segregation were investigated. Of these, two gave promise; the single balanced gate and the double balanced gate. The single balanced gate is recommended because of its simplicity. It is a hand operated swinging plate pivoted under the center of the stream at a point one-third of the way down from the top. This distributes the aggregate in a horizontal layer laterally across the car, thus preventing segregation.—J. E. GRAY

High quality aggregates and value of concrete. PAUL HERTEL. *Zement* (Germany), Nov. 13, 20, 27, Dec. 4, 1930, V. 19, No. 46, 47, 48, 49, p. 1087-90, 1117-21, 1141-4, 1167-9.—Investigations were made as to how quality and costs of concrete can be improved by changing gradation of aggregates according to sieve curves now in practice. Following methods are discussed: (1) Fuller curve takes aggregates and cement into consideration, (2) curve by Herrmann is similar to Fuller curve but gives a greater number of sizes between 0.12 and 0.47 in. and does not consider cement content, (3) Graf emphasizes importance of fine aggregates and gives a favorable mortar curve, (4) Gruen established a void-volume curve. He determines volume of voids of aggregates by filling a cylinder with coarse particles and water and then replacing water by fine aggregate. (5) Otzen derived an ideal curve, similar to Fuller curve which shows directly grain composition of aggregates for certain mixtures, or cement contents per cu. ft. Great number of experiments were made to study effect of improving aggregates by means of above described curves on yield, consistency, weight per cu. ft., compressive strength, flexural strength and water permeability. One blast furnace slag cement and one high alumina cement were used for these tests. In all cases 18.7 lbs. cement were used for 1 cu. ft. of concrete and consistency of all mixtures was plastic. Results show clearly that properties of concrete are greatly improved by such methods, especially water impermeability. Costs of concrete are in general about 20 per cent higher when aggregates are treated in such manners.—A. E. BEITLICH

Control and improvement of the granular composition of sand, pit gravel, and fluvial deposits used in the manufacture of concrete. A. JANOUSEK. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—River deposits excavated for use in concrete have only exceptionally the composition required. To control this composition, it is essential to carry out a grading analysis, which should be executed with all possible care. Attention should be paid to following points: (1) Diameter and geometric shape of grains, in relation to strength. (2) Surface area of single grains, and of total of grains collectively, in relation to strength. (3) Proportion of grains of any one diameter, and relation between various diameters. (4) Means of employing these results for improving granular composition of material. A common in-

direct procedure consists of sieving mixture in a series of sieves, thus measuring the granular composition of a given quantity of material. The surface area of the grains can be determined, although only approximately, by either an experimental or a mathematical method, both of which are based on results of sieve analysis. To use the results of these investigations rationally and economically, it is necessary to organize continuous control on the building site. In addition it is desirable to control mixing of concrete, and to mechanize operations as much as possible with object of eliminating personal influence of incompetent workers. There is at present no sufficiently accurate experimental method for determination of surface area of gravel, but directions are given for calculation of this quantity, a knowledge of which is most important, particularly in connection with tensile strength. In addition the question of international unification of methods of control has been studied, and various propositions elaborated. The essential requirement is a series of sieves having circular holes, .039, .118, .275, .590 and 1.180 in. or .039, .078, .197, .394 and .787 in. in diameter. The natural limits of application of the grading modulus are dealt with, and finally a simple method of comparison of grading analysis curves is suggested for application in the correction of natural grading.

CEMENT

Tube mill grinding. BELLWINCKEL. *Zement* (Germany), Dec. 4, 1930, V. 19, No. 49, p. 1161-3.—Author attacks statements made by A. B. Helbig in previous article (*Zement* (Germany), Sept. 9, 1930, V. 19, No. 37, p. 870-3; JOURNAL A. C. I., Dec. 1930, V. 2, No. 4, *Abstracts*, p. 97), on superiority of tube mills with air separators over ordinary tube mill type. For an economic performance of a tube mill without air separator, the following points must be taken into consideration: (1) Entire length of mill must be divided into 3 or 4 sections, (2) form and weight of steel balls must correspond with increasing fineness of material, (3) separation of coarsest particles by screens and (4) sufficient aeration and cooling of entire mill. These points are discussed for raw mix, coal and cement.—A. E. BEITLICH

Tube mill grinding. A. B. HELBIG. *Zement* (Germany), Dec. 25, 1930, V. 19, No. 52, p. 1234-5.—In reply to article by Bellwinckel (*Zement* (Germany) Dec. 4, 1930, V. 19, No. 49, p. 1161-3), author again emphasizes advantages of air separator installations in tube mills and cites a number of American experiences.—A. E. BEITLICH

Tricalciumsilicate. ENRST JAENECKE. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 26.—After studying paper by L. T. Brownmiller and R. H. Bogue on "The X-ray method applied to a study of the constitution of portland cements," author admits existence of tricalciumsilicate, contrary to his former opinion. Furthermore he points out, that his compound jaeneckeit: $8CaO.Al_2O_3.2SiO_2$ can exist as found by X-ray measurements. He explains necessity of work with petrographic microscope and X-ray in portland cement clinker constitution problem.—A. E. BEITLICH

Cement and concrete in the graphic four-component parallelogram. M. SPINDEL. *Zement* (Germany), Dec. 18, 1930, V. 19, No. 51, p. 1200-5—A four-component diagram in form of a parallelogram is explained and its application advised for proportioning of cement mixtures, especially for high early strength cement. Allowable limit for lime content for manufacture of best cement, as calculated by Kuhl, Jaenecke, Guttmann and Gille are plotted in diagram and compared.—A. E. BEITLICH

Acceptance and control tests for cements. QUIRINO SESTINI. *Ostr. Bauzeitung* (Austria), Dec. 13, 1930, V. 6, No. 50, p. 810-1.—Author claims that ordinary tests do not give a true picture of all properties of cement. Behavior with different aggregates and retarding or preserving agents are not

considered. Opinions of A. J. Poole and C. R. Platzmann on subject of cement testing are discussed. An empirical formula for valuation of cement is derived which gives a relation between compressive strength C and tensile strength T. $R = \frac{T \times 10 + C}{2}$ R is quality factor. (*cf. L'Industria Italiana del Cemento* (Italy), July, 1930.)—A. E. BEITLICH

Determination of soluble silica in puzzolan cements. G. BAIRE. *Chimie et Industrie* (France), March, 1930, V. 24, special number, p. 298-300.—A method is given for determination of soluble silica in puzzolan and mixtures of portland cement and puzzolan or trass. Sample of 1.0 gram is treated with 100 cc. cold water and 25 cc. hydrochloric acid (1.1 per cent). Residue is filtered after gentle heating, washed with hot water and dried. Then it is treated for 20 hr. in the cold and 4 hr. at 122 to 150° F. with 100 cc. of a solution of potassium hydroxide (20 per cent). Precipitate is washed and ignited, treated with sodium carbonate and analysis is then continued as usual.—A. E. BEITLICH

Protective coatings for cement and concrete. *Zeitschr. Ostr. Ingenieur und Architekten Vereins* (Austria), Nov. 21, 1930, V. 82, No. 47-48, p. 408-9.—Principle of most protective materials is formation of an inorganic coating which is more weather resistant and under certain conditions even harder than concrete surface. Fluorides, especially magnesium and zinc fluorides, are most important and effective when an excess of lime is present in cement. Of similar effect are silicate coatings on which a second protective coating is advisable. Silicate esters are used for same purpose lately. Coatings may be white or colored with inorganic pigments. Hardening process of these materials is in general very slowly. Coatings can be made which resist temperatures up to 1100° F. Coatings do not destroy natural porosity of concrete surface.—A. E. BEITLICH

A new method for the determination of consistency of cement mixtures. KARL E. DORSCH. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 26-8.—Inaccuracy of Vicat method for determination of normal consistency of cement-water mixtures led author to design new apparatus which gives more accurate results without use or waste of much cement. Procedure is as follows: 100 g. cement is mixed for 3 minutes with 28 per cent water and then filled into small conical steel cylinder open at top and bottom. (Lower opening is smaller than top opening.) Cylinder with cement is then dropped between two vertical steel rods of special support which has a stop near bottom. When cylinder hits stop a small amount of cement will emerge from bottom opening. Cement which leaves cylinder after 3 runs is collected and weighed in small porcelain crucible. Table shows normal consistency calculated from weights of cement. Method was found to be highly accurate for various cements. Dimensions and drawings of apparatus are given.—A. E. BEITLICH

Temperature and humidity control in cement testing. H. S. MATTIMORE. *Eng. News Record.* Jan. 1, 1931, V. 106, No. 1, p. 36.—Product termed "high-early-strength cement," is more sensitive to temperature and humidity conditions during testing than cement produced several years ago. Specifications allow a tolerance of plus or minus 5 per cent. In some cases this is too broad, as conditions have been experienced in testing high-early-strength cement where a maintained temperature of 65° F. resulted in rejection on strength requirements, whereas these requirements were met when a temperature of 70° or over was maintained. Standard specifications require that relative humidity shall not be less than 90 per cent in moist closet during storage of strength specimens. In Pennsylvania department of highways testing laboratory the special moist air cabinet is constructed with slate interior, 1 in. of cork insulation and steel exterior. Humidity is maintained from 95 to 98 per cent by running water flowing from perforated pipes in each

section down the sides of brass mesh screen mounted ⅜ in. from the side walls. Temperature of water is regulated in temperature control valves prior to entering the cabinet. Temperature records over many weeks indicate a maximum variation of 2°.—D. E. Larson

Comparative investigation of effect of chlorides of calcium, barium and beryllium on setting process and strength of normal portland cement and portland cement without gypsum. K. Fill. *Zement* (Germany), Dec. 11, 18, 1930, V. 19, No. 50-51, p. 1178-84, 1206-9.—One portland cement clinker and one portland cement were mixed with aqueous solutions of calcium chloride, barium chloride and beryllium chloride and changes in setting time, compressive and tensile strength under different storage conditions were studied. Concentrations of solutions were 3, 2, 1, ½, 1/5 and 1/10 molar. Heat tones of reactions of cement and clinker with above solutions were measured and recorded in special insulated apparatus. Effect of these electrolytes on cement and clinker were considerably different, especially in case of $BeCl_2$ solutions. Strength of clinker is only slightly affected, while strength of cement is greatly reduced by $BeCl_2$. Calcium and barium chlorides improve in general strength of clinker and have similar effect as gypsum on setting time. Effect of these salts on heat tones is illustrated in several curves. Work was carried out in the Institute for Mineralogy of University of Frankfurt a. M. (Germany).—A. E. Beitlich

Impermeability and permanence of cements. Edmond Marcotte. *Ciment* (France), Dec., 1930, Jan., 1931, V. 35, No. 12, p. 476-82; V. 36, No. 1, p. 6-12.—The leaking of masonry walls is a subject of much concern and the attack of aggressive waters on concrete is to be connected with penetration of moisture into the concrete body. A complete investigation of the problem would require a very large number of experiments. After discussing briefly the properties of the several French cements the classic work of Michaelis on admixtures is reviewed in detail. The superior results obtained with European pozzolanic materials, especially in increasing resistance to sea water, is emphasized. A cement is now available made by grinding portland cement clinker with *gaize*, a pozzolan found near Paris and containing about 33 per cent alkali soluble silica.—F. O. Anderegg

Reconstruction of cement plant in Rodaun (Austria) *Ostr. Bauzeitung* (Austria), Dec. 20, 1930, V. 6, No. 51, p. 828-30.—New Quarry for raw materials was added to plant. Aerial ropeway delivers material to crushers. After passing dryers raw material enters 2 modern three-compartment mills directly driven by high speed motors and provided with table feeders. Material of any desired fineness can be obtained. Raw mix bins have capacity of 2640 tons. One rotary kiln, designed by Fellner and Ziegler, Frankfurt, is 164 ft. long and 9.8 ft. in diameter; it has an enlarged calcination zone. Kiln is provided with clinker cooler. Waste heat boiler system furnishes necessary electric energy. Gases from kiln and dryers are purified by electric dust precipitator. Entire coal mill was rebuilt and new tube mills installed. Plant manufacturers portland cement, high early strength cement and iron portland cement.—A. E. Beitlich

Special carfloat used to distribute bulk cement. Morris Kind. *Eng. News Record*, Jan. 15, 1931, V. 105, No. 3, p. 111-112.—Bulk-cement delivery in New York City is complicated by fact that while most of cement is shipped from mills by railway, ultimate delivery with but a few exceptions must be by water. To secure quick, safe, economical delivery, bulk-cement cars and float have been constructed and used successfully. Cement is discharged from cars by gravity to a conveyor on the boat and then pumped to shore. There is no loss of cement and no dust. The float is equipped with a Diesel engine, generator, and compressor, and can handle 225 bbl. per hr. with total capacity of 3,500 bbl.—Glenn Murphy

Special and fused cements—two Swedish products. VICTOR BAHRNER. *Teknisk Tidskrift* (Sweden), Jan. 3, 1931, p. 9-11.—These two types of cement that have been used in Sweden for several years, are made of almost same parts. Fused cement is produced as a melting substance. Compressive strength is very high, shown by following table summarized from several charts and diagrams:

	Compressive Strength in Lb. Per Sq. In. After				
	1 day	2 days	7 days	4 weeks	9 weeks
Melt cement	4260	5350	6130	6760	7460
Special cement	1570	2350	3560	4840	5270

The mix was 1:2:4 and the water coefficient 0.6. The time of setting is found to be very short, so the forms can be taken down and reused frequently. The fused cement heats up after pouring. Therefore concrete buildings can be built in freezing weather without being inclosed and heated. Several concrete houses have been constructed during the winters in Stockholm with excellent result. Table gives proportion of fused cement for different temperatures and thicknesses of concrete. It is necessary to keep the concrete from freezing 24 hours. Fused cement can not be mixed with portland cement or lime which will cause immediate set with low strength. The special cement can be mixed in same proportions as portland cement and fused cement concrete can not be made richer than 680 lb. per cu. yd. concrete. It should not be used if the temperature is more than 90° F. As a conclusion it may be said that special and particularly fused cement is suitable for all kinds of concrete construction where high compressive stresses and short construction time are required in freezing weather.—ODD ALBERT

Pre-determination of the composition of the concrete necessary to attain a given compressive strength. R. B. YOUNG. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Method described has been widely used in the United States and Canada. It is based on Abram's water-cement-ratio-strength relationship and the soundness of this law for practical work is indicated. Details of method are outlined and typical data presented. Extensive references to original sources of information are included.—EDITORS N. I. A. T. M.

The degree of lime saturation. HESS. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 28-32.—Degree of lime saturation (S) is described as relation between actual lime content and theoretically possible amount of lime in a clinker: $S = \frac{CaO}{2.8\,SiO_2 + 1.65\,Al_2O_3\,0.7 + Fe_2O_3}$. Replacement of hydraulic modulus in German Standard Specifications for Portland Cement by degree of lime saturation is proposed. Advantages of this new index are characterized and its application for valuation of clinker is clearly illustrated. Index can be easily demonstrated in graphs. Kuhl's method for use of this index is criticised and several improvements are suggested.—A. E. BEITLICH

Effect of storage conditions on the properties of cement. D. A. G. REID. *Concrete Constr. Eng.* (England), Feb., 1931, V. 26, No. 2, p. 155-159.—Investigation deals with effect of storage conditions on the rate of hardening of cements, particularly rapid-hardening cements. Factors investigated are: type of cement (normal portland, rapid-hardening portland, and aluminous); type of container (jute sack, four-ply paper sack, wooden keg, and metal drum); condition of storage (closed shed at normal temperature, cold storage, and oven storage) and period of storage (from 3 mo. to 2 yr). Conditions selected to represent what might actually occur in practice. General conclusions: (1) Loss on ignition increased with storage for all types of cement tested and all containers except the metal drum. (2) Appearance—after severe types of storage (jute and paper sacks and storage in shed) cement appeared coarser to the touch and in some cases air-set lumps were formed. (3) Water requirements for standard consistence rose steadily with increasing

loss by ignition. (4) Fineness—no change except due to air-set lumps. (5) Setting time—not always affected in the same manner or to the same extent. (6) Specific gravity decreased as loss by ignition increased. (7) Decrease in rate of attaining strength.—Joseph Marin

Properties of Concrete

Influence of proportioning of mix and water addition on mortar compressive strength of three cements of different values. Erich Schmidt. *Zement* (Germany), Dec. 18 and 25, 1930, V. 19, No. 51-52, p. 1213-6, 1235-8.—In two extensive series of tests were studied influence of mix (proportions from 1:3 to 1:24) and influence of mixing water (from 4½ to 12 per cent) on mortar strength of one normal portland cement, one high early strength cement and one very high early strength cement (Zenith cement) with river sand. Curves for mortar strength development were in relatively close agreement with those from normal standard tests. Addition of water shows different effect according to age of specimen or kind of cement.—A. E. Beitlich

Watertight brick masonry. F. O. Anderegg. *Contract Record Eng. Rev.*, Nov. 26, 1930, V. 44, No. 48, p. 1476.—In connection with study of watertightness of brick masonry which included tests of cement mortars it was found that ideal grading of sand for mortar is one of uniform increase in amount with size of sieve opening using a standard series of sieves whose mesh sizes differ by a constant factor of 2, e. g. 100, 48, 28, 14, 8, etc. The amount passing a given sieve and retained on the next smaller should be about 1.25 times that retained on next smaller size. For good results brick must have the right range of absorption and a proper surface to bond well with mortar. Cement and lime used in mortar must be (1) suitably plastic to work well under the trowel, (2) adhere very tightly to brick, (3) be sufficiently flexible to take up buffets of wind even in tower buildings, (4) be watertight and (5) be reasonable in price.—G. M. Williams

Recent improvements in making concrete. H. A. La Rue. *Canadian Eng.*, Jan. 20, 1931, V. 60, No. 3, p. 16.—Basic idea underlying most methods of proportioning concrete is that a dense mixture of cement, sand and stone is necessary and that the denser the mixture, the stronger the resulting concrete that is obtained. Various methods are employed in the attempt to attain this maximum density. The so-called void method involves the use of enough cement to fill the voids in the sand to make a mortar and then sufficient mortar to fill the voids in the stone. The amount of cement is usually in excess of that required to fill voids in the sand. The method of mechanical analysis involves a sieve analysis of the aggregate and the addition or elimination of certain sized materials so grading of aggregate will conform to an established grading curve. Account must be taken of the fact that the volume of fine aggregate is influenced by its water content so that to insure the proper amount of this size of material account must be taken of bulking due to presence of moisture which will alter its apparent volume. To produce more uniform concrete it has been established that aggregate quantities should be weighed rather than measured by volume. It is of greatest importance in obtaining high strength concrete to keep the mixing water to the minimum necessary to furnish a workable mix. Sufficient water must be used to result in a plastic workable mix but this necessary amount should not be exceeded.—G. M. Williams

Action of aggressive waters on concrete. Hugo Vierheller. *Bautenschutz* (Germany), 1930, V. 1, No. 2, p. 33-7.—Corrosive action of aggressive waters on concrete may be either chemical, chemical and physical or only physical in nature. These factors must be considered when a concrete is judged

for its suitability for certain purposes. In the first case substances, which are dissolved in water, react with cement portion of concrete and either dissolve or weaken such portions thus changing structure and strength of concrete. Agents, which act in this manner are inorganic acids, some organic acids and acid salts. Protective coatings or drainage of water are advisable. Example for second case is crystallization of new compounds formed by chemical reactions in interior of concrete with considerable increase of volume. Latter fact destroys interior structure and leads to softening of concrete. Pure physical action of water on concrete can be found in cases where water is taken up by capillary action and this water generates osmotic or crystallization pressure. This process can appear without any chemical reaction and takes place in concrete of improper density. Structures only partly immersed in water are most likely to be affected in this way. Action of frost falls in this class. Making of dense concrete with sufficient impermeability is chief preventive measure. Directions are given for systematic investigation of different cements and concrete mixtures and of their stability in corrosive waters.—A. E. BEITLICH

Resistance of concrete pipes to corrosion by water. J. O. ROOS AF HJELMASTER. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—The paper comprises the investigation of typical cases of concrete pipe disintegration in Sweden, laboratory tests of the action of water with aggressive carbon dioxide on concrete pipes of different composition, and the application of a new laboratory method for the determination of the dissolving action of different kinds of water percolating through various kinds of cement.—EDITORS N. I. A. T. M.

Variations of the compressive strength of concrete within the water-cement ratio law. IRA L. COLLIER. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 731-741.—Three different series of tests, covering 450 specimens, were performed to determine how strength of concrete is affected by variation in consistency of mix, and in proportion and grading of sand. Water-cement ratio was varied from 0.6 to 1.4, slump from 1 to 9 in., and percentage of sand from 20 to 80. The cylinders were all tested at 7 days. The tests demonstrated that if water-cement ratio of concrete is held constant, and sand and gravel are in the usual proportions of 1:2, the compressive strength decreases as the slump increases, and decreases as the fineness modulus of sand is increased.—GLENN MURPHY

Mortars and concretes of mixed cements. F. FERRARI. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—The object of this study is to demonstrate relationship between the mechanical (crushing) strength of standard mortar and of concrete of determined composition. Mortars and concretes having as a basis binary hydraulic mixtures (portland cement with other substances, pozzuolanic or inert), possess, at same age, very different strengths, depending principally upon nature and percentage of material added to portland cement.—EDITORS N. I. A. T. M.

Volume change of concrete. C. H. SCHOLER AND E. R. DAWLEY. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 751-766.—This paper treats initial expansion or contraction of concrete during hardening period as well as volume changes after specimen has hardened. Changes in length were measured by an optical comparator consisting of two parallel microscopes. The distance between them was determined by an interferometer reading to 0.00005 in. Readings were taken on pieces of glass tubing inserted in the specimens 20 in. apart. In order to reduce pressure of forms against concrete during setting period, forms were lined with soft rubber slabs. It was found that leaky forms caused a rapid shrinkage until set. This was followed by an expansion for about 24 hr., and then a slow, steady shrinkage for an indefinite period. A water-tight form will eliminate the first shrinkage. It was found that character

of aggregate had practically no influence on volume change. Lean mixes showed slightly less change than rich mixes. Mixes studied were more nearly like pavement concrete than like that used in reinforced building construction. Probably the latter would show the greater shrinkage.—GLENN MURPHY

The elastic behavior of concrete—Uber das elastiche Verhalten von Beton. HIROHIKO YOSHIDA. Verlag von Julius Springer, Berlin, 1930, 114 p.—Work was carried out under direction of E. Probst, Technischen Hochschule, Karlsruhe. Methods are given for determining Poisson's ratio for different types of concrete under static loading, as well as effect of various factors such as changes in stress, age of concrete, water cement ratio, mix, etc. The second part deals with the results of a study of application of frequently repeated load, and gives effect of repeated loads on elastic properties. The last section takes up the elastic behavior of concrete and covers elastic afterworking, elastic potential, and a study of velocity of propagation of load waves in concrete. Conclusions of the author include: (1) The value of μ (Poisson's ratio) for concrete decreases with increase in tension. (2) μ for concrete 1:6 mix under a tension between 142.5 to 570 lb. per sq. in. and 142.5 to 712.5 lb. per sq. in. is about 7. With a 1:10 mix μ for the same tension is about 5.5 to 6. (3) The value of μ increases with repeated load. (4) The relation between value of μ, and the tension, and also the relation between the elastic modulus and tension change with repeated load. (5) The elastic behavior with repeated load is related to the afore mentioned change.—H. G. FISK

Investigation into the strength of mortar and concrete for use in large structures and for the manufacture of cement products. O. GRAF. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—The knowledge requisite for the production of mortar and concrete possessing the greatest resistance to crushing, bending and wear, with great impermeability and the minimum shrinkage and expansion, is, for practical purposes, capable of simple expression. The paper is developed on these lines, while in addition, the factors requiring attention in measuring and working the ingredients are briefly dealt with.—EDITORS N. I. A. T. M.

Cement, concrete and reinforced concrete. Laboratory tests, and control and practice on the building site. A. HASCH. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—An attempt is made to deal briefly, from the present day standpoint, with the relative merits of ordinary and high-strength mortars and concrete, as well as with the tests to which these materials should be subjected in the laboratory and on the building site. The essential point is the organization of control tests on the site, combined with an extension of the laboratory tests, especially on the chemical side, since samples taken before the commencement of building operations have not the slightest value. Reference is made to the statistic (theory of probability) treatment of the results of static tests.—EDITORS N. I. A. T. M.

Flow of concrete under sustained compressive stress. R. E. DAVIS AND H. E. DAVIS. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 707-730.—Data are presented from seven series of tests, starting in 1925, and dealing with 215 specimens. The tests were designed to show how time-flow is influenced by richness of mix, water-cement ratio, character and gradation of aggregate, age at time of loading, magnitude of stress, variation in moisture conditions, length of period of sustained stress, reinforcement, and intermittent loading. In addition, the effects of flow upon strength and modulus of elasticity are studied. Aside from earlier tests, flow was measured over a 10 in. gage length. It was found that rate of flow decreases as time increases. Amount of flow was greater for concretes of a lean mix, with high water-cement ratio, containing aggregates with a small fineness modulus. A high applied stress and air storage also increased flow. Rate of unit stress to total unit deformation was decreased by early loading and air storage, and decreased at a decreasing rate

with increase in time. Sandstone and basalt concretes showed greatest flow, and limestone concrete the least. Quartz, granite, and gravel concretes gave intermediate values. The flow for reinforced specimens was much less as the steel took most of load. Effect of cyclic repetition of stress upon flow is not unlike that for steady loading. The modulus of elasticity of concrete which has been subjected to a sustained stress is higher than that for same quality of concrete not previously stressed. Apparently the compressive strength is not affected. Modulus of elasticity at time of loading is not a criterion of flow.—GLENN MURPHY

Research on the variation of the modulus of elasticity under compression with the composition, fluidity, age and strength of concrete. R. DUTRON. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Modulus of elasticity of concrete, E, is a function of strength, i. e., it depends implicitly on all factors which affect strength: Proportion of cement, nature and grading of inert material, fluidity, and age of the concrete. Value of E may be determined as a function of the crushing strength R of concrete by formula $E = KR^m$; E corresponds to the deformation of concrete under a load equal to three-tenths its breaking load under compression, K and m are two constants which assume different values on the one hand for concrete containing ballast and powdered materials, and on other, for all concretes containing sand.—EDITORS N. I. A. T. M.

The compactness of concrete and its resistance to chemical action. A. POULSEN. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Even the best concrete is destroyed by the reaction of its free lime with the magnesium sulphate contained in sea water. On the other hand, rich mixtures soon show cracks of their own accord. The application of a protective coating to large masses of concrete employed in marine work is, of course, impossible. To neutralize the excess lime it is necessary to add to the cement, pozzuolana containing in sufficient quantity the necessary hydraulic constituents (e. g. SiO_2 soluble in alkalis). The cement with hydraulic admixtures thus obtained should be submitted to laboratory tests. In Denmark, the Danish diatomaceous earth, Mo Ler, which contains about 60 per cent of soluble SiO_2, has been used with considerable success for the past 20 years.—EDITORS N. I. A. T. M.

Method of disintegrating hardened concrete for the determination of the original ingredients. R. LOMAN. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—The object of the work was discovery of a method of determining original ingredients of hardened concrete. It is possible to disintegrate the concrete satisfactorily without smashing up the aggregate. This is done by heating for a time to 800-900 deg. C., and then cooling by suddenly quenching in water. Aggregate is separated from cement and can be sieved; it is almost completely recovered, but has generally become a little finer, owing to ignition and embrittlement on quenching. This increase in fineness is revealed by sieving. Proportion of cement can be estimated by various methods. Experiment has proved, however, that most rapid method, and that which gives most reliable results, consists in heating to 800-900 deg. C., (which removes combined water and carbon dioxide), and then cooling in a space free from water vapor and CO_2. Weight obtained is that of original cement, less its loss on ignition. The method is of ample accuracy for technical work.—EDITORS N. I. A. T. M.

Further studies on the workability of concrete. W. F. PURRINGTON AND H. C. LORING. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 654-667.—This paper is a continuation of a previous investigation of workability of concrete mixtures by measuring power consumed in mixing concrete. Tests were run with a 390 lb. charge in a vertical-axis type mixer at speeds of 24, 30, 35, and 44 r. p. m. Readings were taken 5 min. after water was added. Actual power

consumption was measured by a prony brake. Mixes were designed for 2,000, 3,000, and 4,000 lb. per sq. in. at 28 days. Data tend to show there is an optimum water-cement ratio at which maximum workability of a concrete is obtained. Above and below this ratio a concrete is produced which tends to increase power consumption due in one case to stickiness of particles, and in other to friction of under-lubricated grains. Different brands of cements do not show consistent relations to power consumption when studied over a long period of time. The addition of calcium chloride affects the workability of different cements differently.—GLENN MURPHY

Strength and resistance to sulfate waters of concrete cured in water vapor at temperatures between 100 and 350 deg. F. DALTON G. MILLER. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 636-653.—In order to approximate the thickness of the wall in drain tile used in the public ditches in the Middle West, 2 by 4-in. cylinders were used in this investigation. Although small, the cylinders were essentially concrete, and not mortar. No aggregate exceeded ⅜ in. in size. The mix was 1:3, and the water-cement ratio varied from 0.59 to 0.64. After curing with steam, some specimens were exposed to alkaline waters of Medicine Lake, S. D. Ultimate compressive strengths were obtained at 7 and 28 days, and 1, 3, and 5 years. The cylinders cured continuously in water averaged from 0.5 to 1.3 per cent less absorption than cylinders cured in water vapor. For identical curing periods, the absorption was nearly constant regardless of temperature. Concrete cured in water vapor between 100 and 285 deg. F. and stored in water continues to increase in strength at a rate not greatly different from control cylinders cured in water. Photomicrographs show that steam curing produces more complete hydration. No strength is gained by curing in steam at temperatures above 155 deg. F., or by curing more than 48 hours. Resistance to sulfate waters is definitely increased by curing temperatures from 212 to 285 deg. F., and for periods up to 6 days.—GLENN MURPHY

Deterioration of concrete in hydraulic structures. A. EKWALL. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Inspections 10 years ago of hydraulic structures in Sweden displayed some defects in the concrete. Continued observations proved, that concrete in some cases was subject to an increasing deterioration, apparently due to qualities of concrete materials, to methods of making concrete or to conditions to which concrete was exposed. Investigations were arranged with the aim to ascertaining causes of such increasing deterioration. It had been observed that deterioration occurred only in structures exposed to water on one side and was caused by the solubility of lime and decomposition of other chemical combinations in hydrated cement. Particular attention was therefore placed on solubility of cement and aggressiveness of the water. In combination with investigation relating to these factors, practical experiments have been made with concrete slabs exposed to low water pressure during several years. It was proved that Swedish natural waters need not be considered aggressive in this respect. The importance of using sand, relatively free from organic substances, was impressively established. Leaner mixtures than 1:4½ should not be used for concrete exposed to water pressure from one side. Replacing cement by an admixture of 10 per cent lime or 5 per cent calcium chloride improved the watertightness, while 25 per cent trass or slate powder practically gave no improvement.—EDITORS N. I. A. T. M.

Resistance of concrete to chemical attack. K. MANDL. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—In Austria the deterioration of concrete due to injurious influences is relatively slight. The most prominent instance is the destruction of a pillar of a new dwelling house in Vienna, which was definitely found to be due to the action of gypsum. It is especially noteworthy that this damage was not due to sulphate-containing

ground-water, but to surface water that had percolated through the slag-containing fundamental layer, from which it dissolved the sulphate, being then compelled to pass through the concrete pillar, because the slag deposit was laid on material impermeable to water. Several cases of deterioration of the sewer system of Vienna by sulphate-containing water are also reported. A statement is made regarding an investigation into the action of nitrites on concrete, the results of which have been in part already published. It is not yet, however, possible to draw conclusions from this work.—EDITORS N. I. A. T. M.

Designing concrete for high strength, low permeability and low shrinkage. W. A. SLATER. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—High strength and low permeability depend primarily upon using the lowest possible water-cement ratio consistent with securing workability of the mix. Designing concrete for a given strength and low permeability reduces to combining water and cement in predetermined proportions with aggregates in such proportions as to give workable mixes and low cost of finished concrete. Most important requisite for low shrinkage appears to be low cement content. However, excessive shrinkage has resulted from the use of trap rock or conglomerate screenings as aggregate in mortar or from the addition of integral waterproofing compounds. Moist curing is an effective measure for promoting high strength, low permeability, and low shrinkage.—EDITORS N. I. A. T. M.

The properties of cement-sawdust mortars, plain, and with various admixtures. S. H. GRAF AND R. H. JOHNSON. *Bull. No. 3*, Engineering Experiment Station, Oregon State College, Sept., 1930.—Investigation covering practicability of using sawdust in portland cement mortar includes effect of incorporating sand and certain other materials into the mixtures. From a consideration of the use for which this material may be employed, the more important properties are compressive and tensile strengths and thermal conductivity. From tests on these points and also on resistance of material to certain corrosive agents, and from noting relationships existing between the foregoing properties and the presence of bark, the water-cement ratio, the specific gravity, and the age of the sawdust, the following conclusions are indicated: (1) Safe strengths may be obtained from cement-sawdust mixtures. If bark-free sawdust is used there is a small but distinct improvement in strength. (2) Addition of sand has but little effect on the strength and is detrimental where light weight and the lowest thermal conductivity are important. (3) Ratio, by volume, of total water to cement is fair index of strength. (4) Addition of 1 per cent by weight of powdered diatomaceous earth is of decided advantage. (5) Resistance to corrosion of cement-sawdust compares favorably with that of ordinary concrete. (6) Sawdust which has been exposed to weather for a comparatively short time may be used without material loss in strength. (7) Cement-sawdust material is a better thermal insulator than ordinary concrete but does not have as high strength. (8) In order to obtain adequate strength in the cement-sawdust material, it is necessary to use such a rich mixture (large cement content) that the cost, in spite of the cheapness of sawdust, will be greater than that of ordinary concrete. (9) Special care must be taken in control of amount of mixing water and in placing and curing cement-sawdust material if sound durable product is to be obtained. (10) In any given case, the desirability of using the sawdust-cement material in preference to ordinary concrete will depend on the value placed on light weight and on low thermal conductivity.—IRA L. COLLIER

High quality aggregates and value of concrete. (See MATERIALS—AGGREGATES.)

Acceptance and control tests for cements. (See MATERIALS—CEMENT.)

Impermeability and permanence of cements. (See MATERIALS—CEMENT.)

Special and fused cements—two Swedish products. (See MATERIALS—CEMENT.)

The Ibis bridge at Vesinet. (See ENGINEERING DESIGN—BRIDGES.)

ENGINEERING DESIGN

BRIDGES

Design of a reinforced concrete skew arch. BERNARD L. WEINER. *Proc. Am. Soc. C. E.*, Jan., 1931, V. 57, No. 1, p. 3-112.—The paper includes: (a) A stress analysis based upon the Rathbun method; (b) Theory of proportioning sections; (c) Short-cut for computation of crown stresses in a symmetrical skew "rigid frame" with hinged ends. A numerical example is provided in each instance. Complete derivations of formulas are given for several cases. It is assumed that readers have a working knowledge of the design of a right arch by standard methods.—H. J. GILKEY

North Bathurst street bridge, Toronto. W. D. PROCTOR. *Canadian Eng.*, Dec. 30, 1930, V. 59, No. 27, p. 745.—Steel girder bridge with concrete deck and piers has been completed at a cost of $250,000. Piers consist of four columns 5 ft. by 3 ft. under the cap resting on reinforced concrete footings 17 ft. by 51 ft. which in turn are supported by Westinghouse centrifugal spun reinforced concrete piles jetted into a sandy sub-soil. Concrete deck is carried on four rows of plate girders spaced 14½ ft. o. c. Distance between curbs is 46 ft. with two 6-ft. sidewalks outside. Concrete deck is carried on transverse concrete beams resting on tops of girders and cantilevered out to support sidewalks. Tests of concrete during progress of work showed strengths in excess of 2000 lbs. specified for slabs, piers and abutments and in excess of 1500 lbs. specified for footings.—G. M. WILLIAMS

Design of Box Culverts. G. H. HARGREAVES. *Concrete Constr. Eng.* (England), Jan., 1931, V. 26, No. 1, p. 21-34.—First consideration is that of loadings and resulting bending moments at ends and center of members. Distribution of pressure on base is assumed uniform. Other loads considered are point loads due to wheel loads on the top slab, triangular and uniform loads on side slabs and uniform loads on top slab. Maximum corner moments occur when horizontal slabs are subjected to their maximum loads. Calculation of moments at ends of slabs of a box culvert are simplified by use of diagrams. Bending moments are first expressed in terms of a constant F multiplied by a function of the load on the member and width or height of culvert. Diagrams are given for each type of loading mentioned. The values of the constant F for a selected ratio of moment of inertia of top and side slabs are obtained from the diagrams for ratio of height to width used. Moments thus obtained for different types of loading (a diagram is given for each loading) are then correctly combined to obtain maximum moments. Diagrams are given from which the coefficient F can be obtained for: uniform distribution of load on the base slab combined with concentrated loads on the top slab; uniform distributed load on top and bottom slabs; uniform side loads; triangular side loads. Maximum stresses may be determined in usual way neglecting effect of direct thrust. An illustration shows typical arrangement of main steel. Calculations are given for maximum moments in a box culvert of known dimensions and assumed loads.—JOSEPH MARIN

The Ibis bridge at Vesinet (Seine-et-Oise). A. MESNAGER, F. BARTHES, J. VEYRIER. *Genie Civil*, Jan. 17, 1931, V. 98, No. 3, p. 61-65.—A footbridge rebuilt for automobile traffic presented the difficulty of a minimum clearance

above water level of 6.2 ft. in a 29.5-ft. arch span. A rise of 9.8 in. in 14.1 ft. permitted all vehicles to pass. Novel features of the bridge were steel pipes filled with concrete, which constituted the arch. This "integral reinforcement" offers better resistance to transverse expansion of concrete due to longitudinal compression. Tests of concrete surrounded by a shell of steel were made in 1915 by Rabut at the Testing Laboratory of the École des Ponts et Chaussées and by Professor Baes of Brussels, Belgium, reporting at a convention in Vienna in 1928. Messrs. Mesnager and Veyrier patented an adjustable end connection (a screw stopper) eliminating all danger of loss of strength in tubular reinforcement by contraction. At the beginning of 1930 tests made by the Laboratory of the City of Paris on tubular steel specimens filled with concrete, yielded results resembling those of a theoretical homogeneous specimen, whose Young's modulus, elastic limit and compression strength lie half way between those of steel and concrete. An allowable stress of 2845 lb. per sq. in. is admissible over the entire section, when the ratio of steel to gross section is 0.09 and the weight of steel is 1115 lb. per cu. yd. of total volume. In accordance with the steel ratio, the allowable stress used in the design of bridge was 4980 lb. per sq. in. Anchorage of tubular reinforcement is designed with great care. In filling the shells with concrete, pneumatic hammers produced vibration in the mass leaving the hopper. Concrete mix was proportioned as follows: 1.05 cu. yd. gravel (passing mesh with .39 in. diameter opening); .51 cu. yd. river sand; 880-lb. of Supercilor (special) cement and 31.2-gal. of water. (Supercilor cement requires considerably more water than ordinary cement.) The strength varied with the type of mold used for the specimen, being an average of 4550 lb. per sq. in. for screwed molds and only 2700 lb. per sq. in. for nailed molds, due to a defect. The tensile strengths were 597 and 234 lb. per sq. in. respectively for the two types of molds. Bridge was tested on August 12, 1930, with uniformly distributed load of 815 lb. per sq. ft. and a 10-ton roller.—M. A. CORBIN

Testing concrete bridges. C. S. CHETTOE. *Concrete Constr. Eng.* (England), Jan., 1931, V. 26, No. 1, p. 5-8.—Reasons are given why the deflection tests of concrete bridges should not be the same as for steel bridges (namely a deflection limit of 1/1600 of the span length as the maximum deflection). Loading and deflection tests for concrete bridges should be made at least one year after the completion of the bridge for following reasons: (1) Concrete gains strength with time and the working stresses for concrete are generally based on strength at 90 days. (2) Temperature stresses are more important in concrete bridges than in steel bridges. (3) Cracks that would not otherwise appear, may be produced by deflection tests made before 90-day period. (4) Most of shrinkage of concrete occurs during 90-day period. The particular effects produced in different types of bridges are discussed. In beam and slab bridges temperature changes and resulting stress in the steel may be worth consideration, though effect on concrete is negligible. Shrinkage and plastic effects are discussed. Temperature and other factors in frame bridges are in general slightly more pronounced than in slab or beam bridges. In fixed and two-pin arches the dead load and shrinkage stresses alone will produce plastic yield, as those due to live load will only be applied for short periods and those due to temperature will be subject to reversal. Temperature and shrinkage stresses become of greater importance the flatter the arch. Three-pin arches, if pin friction be neglected, are unaffected by temperature and shrinkage, and are only subjected to local effects of plastic yield. It is evident that almost all concrete bridges vary periodically in strength due to temperature changes, and that they are subject to a progressive strengthening due to the gradual increase in the strength of concrete. Secondary effects due to plastic yield in general, relieve concrete of some stress as time goes on. Usually shrinkage will have little influence after three months. In conclusion, concrete bridge should not be subjected to loading and deflection tests at its completion but tests should be made at least one year after the completion.—JOSEPH MARIN

The International contest for the Dreirosen bridge in Basel (Switzerland). LEOPOLD BERGER. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 59-61.—Designs and costs of proposed structures of steel and reinforced concrete for the Dreirosen bridge over river Rhine in Basel are discussed. Designs were asked for bridge with one or three openings, center opening being at least 328 ft. wide. Costs for reinforced concrete bridges are in general only 9 per cent higher than prices of steel structures.—A. E. BEITLICH

A method of arch design. G. P. MANNING. *Concrete Constr. Eng.* (England), Feb., 1931, V. 26, No. 2, p. 129-141.—To estimate dead load and temperature stresses in an arch without a number of approximations, it is well to calculate beforehand the approximate area required at crown section. It is also useful to know saving effected by using richer mix of concrete and, for an open spandrel arch, best ratio of breadth to depth for the ribs. To obtain uniformity of elastic properties under varying stresses, examination of strength of cross sections under combined bending moment and thrust shows that about 1 per cent steel top and bottom is good assumption. For preliminary investigation it is sufficiently close to assume stress $= \frac{thrust}{area} \pm \frac{moment}{modulus}$ for the case where tension occurs. Example is given of parabolic arch of 150-ft. span and 30 ft. rise. Curves given show relation between approximated temperature stresses (including temperature crown drop stresses) and thickness expressed as a ratio of length. These curves are given for different ratios of rise to span length for both the extrados and intrados and for a rise or fall in temperature of 30 deg. F. Stresses calculated from relation expressed above. Calculations show that temperature stresses are almost independent of the arch profile and practically independent of area of reinforcement (since increase in steel increases moment of inertia in same proportion). For selected arch a diagram shows relation between approximate dead load stresses at crown for uniform dead load and thickness at crown expressed as a ratio of the span length. Curves are given for both the extrados and intrados and for different ratios of rise to span. Increasing the reinforcement from 2 to 4 per cent reduces the stresses about 20 per cent. Similiar approximate curves for stresses at crown section are given for stresses due to a spandrel load. Increasing the reinforcement from 2 to 4 per cent decreases the stresses about 20 per cent. Stresses due to live load are obtained by using the influence line to first obtain values of moment and thrust for crown section. Corresponding crown-drop stresses are readily found from diagram similiar to temperature stresses. Increasing reinforcement from 2 to 4 per cent reduces live load stresses about 20 per cent. Examples apply above diagrams to find thickness at crown of filied-spandrel and open-spandrel arches of given span and rise. The section required is determined for different allowable unit stresses and most economical mix of concrete thus determined.—JOSEPH MARIN

BUILDINGS

The Ford factory at Dagenham. *Concrete Constr. Eng.* (England), Jan., 1931, V. 26, No. 1, p. 8-19.—In factory which will eventually be largest motor car factory in Europe, main block of office buildings is to be 200 ft. long by 50 ft. wide. Three main buildings comprise the foundry and machine shop, the manufacturing building and the assembly shop. A reinforced concrete tunnel 8 ft. 6 in. high and 12 ft. 6 in. wide extends for more than 1000 ft. and carries the power, heating, oil, lighting, and water service mains. Ultimately two blast furnaces of a 24-hour nominal capacity of 500 tons will be in operation. There are to be 45 coke ovens capable of carbonizing 800 tons of coal in 24 hours. A 30,000 k. w. turbine running at 3,000 r. p. m. will be installed in new power house. An important unit is the ore yard for storage and handling of the coal, iron ore and limestone. Work done during the past sixteen months

includes: excavation, 500,000 cu. yd.; piles driven, 13,000; steel work erected, 5,500 tons; reinforced concrete laid, 150,000 sq. yd.; length of railway track laid, 3½ miles; bridges constructed, 2; length of roads constructed on embankments, 1½ miles.—JOSEPH MARIN

DAMS

Third power development in Stave-Alouette group. *Contract Record Eng. Rev.*, Nov. 26, 1930, V. 44, No. 48, p. 1471.—Ruskin hydro-electric power plant of the British Columbia Electric Railway Co., consists of concrete lined tunnels, steel framed power house clothed with concrete and concrete dam of overflow type. Tunnels are 24 ft. in diameter through solid rock and lined with concrete. Dam is of the gravity type 400 ft. long on the crest with a total height of 185 ft., surmounted by concrete piers carrying a roadway, with the openings between piers closed by Tainter gates 33 ft. wide and 26 ft. high.—G. M. WILLIAMS

Construction of Ruskin power project. C. E. BLEE. *Canadian Eng.*, Jan. 13, 1931, V. 60, No. 2, p. 13.—British Columbia Power Corporation, Ltd., has recently completed a power development on Stave River near Ruskin, B. C., including concrete diversion dam of overflow, gravity type, straight in plan with a crest length of 195 ft. and height of 195 ft. above lowest foundation point. On spillway crest piers are 40 ft. o. c. carrying a roadway with Taintor gates 33 ft. by 26 ft. installed between piers to regulate discharge. Dam is designed to have resultant pressure fall within middle third with reservoir full with uplift varying from 50 per cent head at upstream face to 25 per cent at plane of drainage system, and zero at the downstream toe. Total concrete in dam is 105,000 cu. yd. Field mix for body of dam was 1 cement, 3.05 sand, 2.69 gravel (3 in. to 6 in.) which gave about 2,000 lb. per sq. in. at 28 days. Concrete weighed 155.6 lb. per cu. ft. Cement requirement was approximately 1 Canadian barrel per cu. yd.—G. M. WILLIAMS

Some new ideas on dams. EDWARD GODFREY. *Struct. Eng.* (England), Jan., 1931, V. 9, No. 1, p. 11-17.—Practically every failure of a dam, whether of masonry, concrete, or earth, can be explained by underpressure and neglect to provide for underpressure in design. Underpressure at present is rarely emphasized. Only possible reduction of underpressure, assuming either the thinnest fissure, saturated sand or saturated earth beneath a dam, is reduction due to escape of water. For a dam founded on solid rock the possibility for entrance of water, and consequently safety of the structure, depends upon perfection of construction. Escape of water below dam is usually prevented by plugging leaks, thus causing water under dam to accumulate its full head. Where a dam fails by underpressure it may be a sliding failure due to lubricated bottom. Erosion of soil may also follow underpressure. Failures of St. Francis dam in California, Gleno dam in Italy, and Lafayette earth dam in California are explained by consideration of underpressure. Various other illustrations of failures are cited as evidence of the points raised.—V. P. JENSEN

Recent advances in buttress-type dams. CALVIN V. DAVIS. *Civil Engineering*, Feb., 1931, V. 1, No. 5, p. 387-391.—As a result of recent advances in structural design, two distinct types of buttress dams have been developed, characterized respectively by massive buttresses and by thin buttresses. Principal characteristics of thin buttress type are use of minimum buttress thicknesses and spacings, and placing of greater dependence on stabilizing effect of water load on deck than in massive buttress type. Very low sliding factors may be obtained making dam suitable for almost any type of foundation. On soft foundations, limiting feature is span of slab footing between buttresses. Better foundation design is obtained by keeping buttresses

relatively close together thus necessitating thin buttress design. Economy may be obtained by use of massive buttress type with good foundations. New design with round-head buttresses has following advantages: (1) Water loads transmitted in direct compression to buttress through massive cylindrical portion. Radial water bearing face, to which all pressures are normal, eliminates bending and diagonal tension in upstream portion. (2) Shape of buttress gives almost ideal distribution of principal stresses and shear throughout dam. (3) All members are massive and solid. Volume of concrete in massive buttress type is about one-third of that in gravity type, in which governing feature is safety against overturning. Reduction in cost of massive buttress type over gravity type is about 50 per cent for dam 300 ft. high. Aside from development of two types mentioned, most noteworthy advances in design and construction have been improved methods of stress analysis and development of system of buttress construction and contraction joints. Modern methods of stress analysis were introduced in this country 20 years ago by Professor Cain and have been recently adapted to design of buttress-type dams. A more thorough knowledge of stress distribution has been directly responsible for development of improved system for placing construction joints in high buttresses. In laying out system of joints for buttress it is desirable to retain monolithic action of buttress as a whole and to place joints so there will be no tendency for relative movement of buttress sections separated by joint.—D. E. LARSON

MISCELLANEOUS

Discs and disc sectors. *Ostr. Bauzeitung* (Austria), Dec. 20, 1930, V. 6, No. 51, p. 819-22.—Mathematical formulas are given which illustrate nature of discs and disc sections and their importance in static calculations of structures. (cf. *Ostr. Bauzeitung* (Austria), 1930, V. 6, No. 20 and 40.)—A. E. BEITLICH

Introduction to calculations of frame structures. *Ostr. Bauzeitung* (Austria), Dec. 27, 1930, V. 6, No. 52, p. 836-42.—Many tables are used to explain fundamental principles of calculations of structural members. Subjects of discussion are: degree of indetermination of structural members, laws of symmetrical loads, deformation of simple frames and deformation under consideration of torsion and bending effects.—A. E. BEITLICH

Harbor enlargements in Boulogne sur mer (France). BOLLE. *Bautechnik* (Germany), Oct. 3, 1930, V. 8, No. 43, p. 658.—Extensive construction program for enlarging existing harbor structures includes breakwater 1.24 miles long, extension of another breakwater about 2300 ft., excavation of harbor entrance and western part of harbor to a depth of 42.7 ft., quay for passenger service which will be 984 ft. wide, construction of railway station and dock 2300 ft. long.—A. E. BEITLICH

Structural tests on models by Begg's method. J. BLAZEK. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The author considers the relative advantages and disadvantages of various methods of solving problems in statics by tests made on models; in particular the following: (1) The method of exact reproduction on a reduced scale. (2) The method of photoelastic measurement. (3) The Nupubest method. (4) The Continostat method. (5) Begg's method. The author finally reaches the conclusion that Begg's method is most suitable in practice, provided the accessories are well adapted to their purpose.—EDITIONS LA TECHNIQUE DES TRAVAUX

Coefficient $n = \frac{E_s}{E_c}$ and allowable stresses for concrete and steel in reinforced concrete. M. ROS. *Reports*, First International Congress for

Concrete and Reinforced Concrete, Liege, 1930.—Value of n is not fixed by uniform methods in specifications used in different countries but varies from 5 to 20. Value of maximum compression in concrete as calculated differs from value obtained using n. Hence, following points were investigated: Actual properties of portland cement, increased strength of concrete, values of maximum stresses corresponding to change in value of E_c. Influence of change in value of n on stress in concrete was studied—for stress in steel of 14,250 lb. per sq. in. and 1 per cent steel, maximum compression for concrete = 1050 lb. per sq. in. for $n = 5$, and 615 lb. per sq. in. for $n = 20$. Study was next made of actual values of n for: (1) Flexure—Value of n depends upon the loading. Let M_a = the external moment, M_i = internal calculated moment. Assuming Hookes Law and neglecting tension in concrete, maximum compression in concrete $= \frac{2M_a}{x(h-x/3)}$ and the maximum tension in the steel $= \frac{M_a}{J_c(h-x/3)}$, where x = the distance from outer fibre in compression to neutral axis and h = depth of beam. But x is a function of n therefore $x = nf_c\left(\sqrt{1+\frac{2h}{nf_c}}\right)-1$, where f_c = stress in steel. Values of maximum stresses are thus dependent upon value of n. From stress-strain curves of concrete and steel, stresses for a given section of beam for known load and percentage of reinforcement can be obtained. Internal moment can be calculated assuming plane sections remain plane. With above relations actual values of n can be obtained, using stress-strain curves. Choosing arbitrary values of n the values of stresses in steel and concrete can be calculated for given load. These can then be compared with actual corresponding values of stresses. This method was applied to beam of rectangular section with 1½ per cent steel and subjected to flexure. Results are given by curves, one for each concrete considered for beams with concrete of compressive strength of 2,000, 3,250 and 4,300 lb. per sq. in., with corresponding steel of apparent elastic limit of 38,500, 38,500 and 50,000 lb. per sq. in. respectively. For case 1, n varied from 8.3 to 35 corresponding to stresses in concrete from 0 to 2,000. The calculated compressive strength for $n = 10$ is less than, or equal to allowable compression of 930 lb. per sq. in. For values of stresses greater than allowable there is a difference between actual compressive stress and that calculated for $n = 10$, increasing as stresses become greater. Actual stresses in steel are little different from calculated values, using $n = 10$. For concrete of compressive strength = 3,250 lb. per sq. in., n varied from 6.7 to 13.6. Value of n at failure of beam = 7.2. Good agreement was noted between actual and calculated values of stress in concrete using value of $n = 10$. The ultimate strength as calculated using $n = 10$ and compression in concrete = 3,250 lb. per sq. in. agrees well with the actual value. For concrete of compressive strength = 4,300 lb. per sq. in. n varied from 5.8 to 9.5 corresponding to stress in concrete from 0 to 4,300 lb. per sq. in. Value of n at failure = 6.6. Actual values of compression in concrete are a little higher than those calculated for $n = 10$. For tension in steel, actual values are a little smaller than those calculated for $n = 10$. Strength calculated using $n = 10$ and compression in concrete = 4,300 lb., is a little greater than actual value. At failure by flexure $n = 20$. Actual values of tension in steel are less than calculated values for $n = 10$. (2) Axial compression—Values of n were determined for given load by means of stress-strain curves for three cases, strengths of concrete of 2,000, 3,250 and 4,300 lb. per sq. in. with strength of reinforcing steel of proportional limit = 27,100, 27,100 and 38,500 lb. per sq. in. respectively. For strength of concrete of 3,250 and 4,100 value of n varied from 5 to 12. For these two cases the value of $n = 10$ gives close agreement between calculated and actual values of stresses. For 2,000-lb. concrete, n varies from 8 to 18. Following values are proposed for new Swiss specifications for strength of reinforced concrete, discussed by Swiss Society of Engineers and Architects. (1) Coefficient n is to be taken as 10 for compression and flexure. This value of n gives close agreement between the actual and calcu-

lated values of tension in steel and compression in concrete. (2) For $n = 10$ the values of compression in concrete are 10 to 25 per cent greater than obtained for $n = 15$ or 20. Allowable values are thus increased, corresponding to strengths of concrete cubes in simple compression. (3) Allowable shearing stress is between 57 and 71 lb. per sq. in. Quality of concrete is specified. Allowable values of stress in reinforcing steel is 17,000 and 23,000 lb. per sq. in. for steel whose apparent elastic limit is 39,000 and 50,000 lb. per sq. in. respectively. Allowable stresses in steel and concrete for columns and beams are given.—J. MARIN

Calculation methods. "Bemessungsverfahren." B. LOESER. 1930, Wilhelm Ernst and Son, Berlin (Germany), third edition, R. M. 6.50. Reviewed in *Zement* (Germany), Dec. 4, 1920, V. 19, No. 49, p. 1170.—Book prepared for contractor and designer, explains large number of different calculation methods. Special consideration is given to changes of certain dimensions and the effect of such changes on other structural parts.—A. E. BEITLICH

Reinforced concrete construction. Its theory and application. "Der Eisenbetonbau. Seine Theorie und Anwendung." E. MOERSCH. 1929, Konrad Wittwer, Stuttgart (Germany), V. 1, second part, sixth enlarged edition, 541 pp., 614 illustrations, R. M. 27.00. Reviewed in *Zeitschr. Ostr. Ingenieur und Architekten Vereins* (Austria), Dec. 19, 1930, V. 82, No. 51-52, p. 439.—Entire theory of reinforced concrete construction is explained based on newest research developments. Numerous experiments aid in explanation of subject.—A. E. BEITLICH

Increasing of strength of steel skeleton structures with concrete encasements. FRITZ EMPERGER. *Zeitschr. Ostr. Ingenieur und Architekten Vereins* (Austria), Nov. 21, 1930, V. 82, No. 47-48, p. 406-8.—Author discusses results of tests carried out mostly in United States on structural steel encasements. (cf. Morris and Shank, "Gunite and concrete encasement to increase strength of structural steel," 1928, Columbus, O., and reference list, R. Zipprodt, *J. Western Soc. Eng.*, 1929, No. 5.) Great advantages of this type of encasements are illustrated and their economic importance characterized. Number of profiles of steel girders can be reduced. Static calculations of such structures are given. Similar tests are carried out by Prof. Klockner in Prague (Czecho-Slovak Republic).—A. E. BEITLICH

Handbook for reinforced concrete construction. "Handbuch fuer Eisenbetonbau." H. C. EMPERGER. 1930, Wilhelm Ernst and Son, Berlin (Germany), Vol. 1, fourth edition. Reviewed in *Zement* (Germany), Dec. 4, 1930, V. 19, No. 49, p. 1170.—The new edition of Vol. 1 deals with historical development, experiments and theory. Prof. Foerster wrote first chapter. Experiments carried out by Otto Graf are described and widely discussed. Chapters on statics are treated by Prof. Domke. Results of and experiments with different methods are illustrated and relations between theory and practice are described. Numerous references are included.—A. E. BEITLICH

A practical venturi meter for irrigation service. J. E. CHRISTIANSEN AND I. H. TEILMAN. *Eng. News Record*, Jan. 29, 1931, V. 106, No. 5, p. 187-188. A Venturi meter made of precast concrete tube has been developed to eliminate difficulties common to usual devices used to measure irrigation water. About 50 of these meters ranging in size from 16 to 42 in. were placed in service in the Consolidated Irrigation District near Fresno, Calif., during 1929-1930 irrigating season and were found to be practical for actual field use. Meters consist of three sections of precast concrete pipe to form the tube and a vertical well for measuring heads. Beginning at the upstream end, parts of meter consist of: (1) a short entrance section of uniform diameter containing the entrance pressure connections; (2) combined entrance cone and throat sections with piezometer ring, and (3) exit cone of gradual taper providing a

return to original diameter. Sections are cast in heavy sheet metal forms and reinforced with ¼-in. bars. Installation of tube is similar to manner of installing same amount of concrete pipe.—D. E. LARSON

Structural design and construction of concrete improved in 1930. *Concrete*, Feb., 1931, V. 38, No. 2, p. 16-18.—Conviction that high quality in concrete means density as well as strength has spread in past year. Interest is manifested in use of large size aggregates in massive work, particularly dams, mixers using up to 9-in. sizes. Concrete containing large sizes was tamped or puddled by means of electric-driven tampers or vibrators. Most important development in waterproofing is investigation at Bureau of Standards, of 75 commercial products. Railway engineering association developed formulas for design of flat-slab structures under concentrated loadings. Reinforced concrete frames are being built to greater heights, column sizes being reduced by use of cores of structural steel H-beams or cast iron centers. Light-weight aggregates reduce weight of structural frame and floor. Value of continuous belt conveyor on large project demonstrated in construction of dam near Wilmington, Del. Concrete was successfully transported a distance of 1900 ft. from mixer to deposit. Use of and development of equipment for handling of bulk cement is advancing. Revised specifications for placing of terrazzo floors under preparation by terrazzo association. Use of metal forms in building construction evident.—C. BACHMANN

The Nag Hamadi barrage. *Engineering* (England), Jan. 2, 1931, V. 131, No. 3390, p. 20.—The Nag Hamadi barrage, below Luxor in Egypt, the latest of a series of works built across the Nile to increase facilities for irrigation in Upper Egypt, consists of two abutments, nine main piers each 13 ft. 1 in. wide and 90 intermediate piers, 6 ft. 6 in. wide, supported on a concrete foundation approximately 10 ft. thick, of 1:6 portland cement concrete paved with gravel at the sluice openings. Piers are of limestone ashlar with rubble core, the lower portions being faced with dressed granite.—G. M. WILLIAMS

Statics. Part 1. Fundamental principles of statics of rigid body. "Statik. Teil 1. Die Grundlagen der Statik starrer Koerper." FERDINAND SCHLEICHER. 1930, Walter de Gruyter and Comp. Berlin and Leipzig (Germany), 143 p., 47 illustrations, R. M. 1.80. Reviewed in *Zeitschr. Ostr. Ingenieur und Architekten Vereins* (Austria), Nov. 21, 1930, V. 82, No. 47-48, p. 412.—Chapters of book deal with: introduction, forces with one point of attack, graphic demonstration of forces in the plane, force couple and moment, principle of virtual energy and equilibrium conditions. Extensive reference list is given.—A. E. BEITLICH

Statics of structural members. "Statik der Tragwerke." WALTER KAUFMANN. 1930, Julius Springer, Berlin (Germany), second enlarged edition 322 p., 368 illustrations, R. M. 19.50. Reviewed in *Zeitschr. Ostr. Ingenieur und Architekten Vereins* (Austria), Dec. 5, 1930, V. 82, No. 49-50, p. 424.—Six chapters of book deal with (1) general fundamental principles; (2) moments, transverse and normal forces in statically determinate structures; (3) determination of tensions in statically determinate structures; (4) elastic deformations; (5) theory of statically indeterminate systems and (6) statically indeterminate structures. Book is a comprehensive treatise of this subject. Method of S. Mueller for solution of general elasticity equations with only one unknown is described.—A. E. BEITLICH

New quays in the harbor of Hamburg (Germany). *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 39-40.—New quay wall for Rhenania-Ossag Oil Co., in Hamburg was built during winter 1929-30. Reinforced concrete foundation piles are from 39.7 to 45.9 ft. long and 13.4 in. square. Steel sheet pilings of Larrsen type No. 3, which are 38.7 ft. long, were used. Top of wall is faced with granite slabs. Concrete piles are provided with acid repellent coating. Tracks of travelling crane are supported by wall.—A. E. BEITLICH

Stresses and strains in spirally reinforced columns. F. E. RICHART. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930. In studies of concrete subjected to compression in three directions, it was found that the material in a spirally reinforced column obeyed the same laws as a plain concrete cylinder loaded at the ends and subjected to lateral pressure. Determination of pressure exerted by spirals upon the concrete core required accurate knowledge of stress-strain properties of the steel, together with a comprehensive series of strain measurements on the columns. The plastic condition of the concrete of the column core at high loads was well shown by the flow of concrete past the spiral reinforcement and by properties of the concrete alone after it had undergone large deformations.—EDITORS N. I. A. T. M.

ARCHITECTURAL DESIGN

Church construction "Evangelischer Kirchenbau." 1930, Editor: Beratungstelle fuer kirchliche Kunst beim evangelischen Presseverband fuer Rheinland in Essen (Germany). Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Uses of reinforced concrete and steel in modern church construction are illustrated in comprehensive manner and several examples are given. Development of modern church type in Europe is discussed from viewpoint of the architect.—A. E. BEITLICH

St. Nikolai, a church of reinforced concrete and glass. PAUL GIRKON. *Zement* (Germany), Jan. 1, 1931, V. 20, No. 1, p. 1-24.—Principles of construction and architectural design of an outstanding example of modern church are entirely new. Only glass and reinforced concrete were used. Calculation of frames of east room against wind pressure was very difficult. This room is formed by three glass walls, structural members of which are made of reinforced concrete. Only two 12 by 12-in. columns are erected in corners. Tower structure is entirely separated from main building. It is 167.3 ft. high and has box-shaped cross-section, 21.7 ft. wide, walls being 7.9 in. thick. Horizontal girders (31.5 by 7.9 in.) are arranged 29.5 ft. o. c. They separate tower walls into single reinforced concrete panels 21.7 by 29.5 ft. Foundation of tower consists of reinforced concrete ring 6.6 ft. wide and 27.6 in. thick; it is placed 21.3 ft. deep. Greatest part of article is devoted to consideration of architectural design and characterizes suitability of reinforced concrete for such construction purposes, with detailed description of different parts of interior. Arrangements of lights show entirely new features.—A. E. BEITLICH

FIELD CONSTRUCTION

BRIDGES

Three cableways and precast units speed winter bridge construction. B. A. WILDER. *Eng. News Record*, Jan. 8, 1931, V. 105, No. 2, p. 64-65. —Winter Construction on a $300,000 concrete arch road bridge across Androscoggin River in Maine was made more certain by pre-casting the 250 curtain walls. They were placed by means of cableways—a separate one for each of the three arch ribs.—GLENN MURPHY

BUILDINGS

Concrete structural frame climbs three stories per week. CEDRIC WILLSON. *Concrete*, Feb., 1931, V. 38, No. 2, p. 41-42.—Schedule on 15-story concrete frame, Oklahoma building, calling for 3 floors per week, was easily maintained. By getting high early strength from standard portland cement, contractor was able to use two shifts, and maintain time schedule without

trouble or extra expense. All concrete mixtures were designed by strength charts. Three classes of concrete were used, containing 6, 7, and 8 gal. water per sack of cement. Mixes were designed to contain 6½, 5½, and 4¾ sacks of cement per cu. yd. of concrete. Four sets of forms were used, being stripped in 48 and 72 hr. and 7 days. Test cylinders broken at 3, 7, and 28 days showed high strengths.—C. BACHMANN

New system of construction. *Conc. Building Conc. Products* (England), Dec., 1930, V. 5, No. 12, p. 153.—Hollow precast units are used in the system described, for walls, columns, beams and floors, etc., and it is claimed that the system is adapted for rapid construction, architectural treatment without extra cost, and prevention of condensation. It is cheaper than steel or reinforced concrete. For walls units are standardized in sizes for various thicknesses, with grooves at each end for grouting, and vertical cavities. Face is 2 in. thick granite concrete and the rest is clinker concrete. For columns the vertical cavities are reinforced with steel and concrete and these stanchions may be part of the wall or stand out from face. Beams are formed in precast units used as shuttering, and which form a trough into which concrete and steel may be placed. These units may be granite faced for use as external beams. At junctions with columns, holes are made in beam shells to allow the column steel to pass through. Floor slabs are made in 2 ft. by 18 in. by 8½ in. units with two longitudinal cavities. The sides of slab and webs between cavities are 1 in. thick. Steel and concrete may be placed between adjacent slabs thus allowing beams to be formed within the floor construction. A wide range of cast stone units may be used in conjunction with the system for decorative purposes. These are cast in strong wooden frames divided with metal dividers. Wing nuts and bolts through gas barrel are employed to hold the whole rigidly together. Each mould makes 14 blocks and they are erected in tiers, five high. Concrete mixers are fed by materials from overhead bins, charged by elevators from stock piles outside the building.—JOHN E. ADAMS

DAMS

The Don Martin project. ANDREW WEISS. *Proc. Am. Soc. C. E.*, Dec., 1930, V. 56 No. 10, p. 2141-2160.—Don Martin Project, one of the largest constructed under the Mexican Federal Irrigation Law of 1926, is located on the Salado River in the states of Coahuila and Nuevo Leon. The reservoir covers 48,412 acres, will store 1,123,600 acre ft. and will serve 160,000 irrigable acres. The dam (41 miles upstream from Rodriguez) is a composite concrete and earth structure. The overflow (concrete) section is 768 ft. long, is of the round-head buttress type and about 86 ft. high to spillway crest. The aggregate was crushed limestone from the right abutment. Part of the same material was returned through cone crushers to supply the sand. The concrete plant consisted of two 1 cu. yd. mixers located directly below the storage bins. Concrete was conveyed in side-dump cars. The 3231 linear ft. of earth and gravel embankment section was protected by a concrete slab varying from 11 in. at base to 8 in. at crown reinforced with ¾ in. round bars 12 in. o. c. each way.—H. J. GILKEY

Toro Rapids dam. *Contract Record Eng. Rev.*, Dec. 10, 1930, V. 44, No. 5, p. 1517.—The Shawinigan Water and Power Co. has completed an impounding and regulating dam on the Mattawin River in Northern Quebec located 80 miles from a railway. The dam consists of a bulkhead section of gravity type, a sluiceway section and a control gate section. The Toro dam is 90 ft. high, 2400 ft. long at the crest, including 1700 ft. of earthfill and contains 52,000 cu. yd. concrete. Concrete aggregates consisted of sand excavated from below water level and stone quarried and crushed near the dam site. An inundator was used in proportioning and concrete was distributed by means of a tower and cableway.—G. M. WILLIAMS

Foundation procedure at Owyhee Dam. *Eng. News Record*, Jan. 29, 1931, V. 106, No. 5, p. 178-182.—Foundation problems, complicated by the necessity of excavating and concreting a fault zone 190 ft. below streambed, have been solved at the 530-ft. arched-gravity dam being built for the Owyhee project in eastern Oregon, and concrete placing in the main structure is well started. It was necessary to remove 38,000 cu. yd. of material from 250-ft. length of fault zone. This was followed by replacement with concrete. Completion of this work permitted concreting operations on a full scale, and about 1,000 cu. yd. per shift are being placed in the structure from an 8-cu. yd. bucket handled from an overhead cableway. The mix is proportioned to secure finished concrete having economic balance between qualities of strength, impermeability and durability, with due regard for desirability of using all the coarse aggregate brought to the site. Result of this balancing of concrete characteristics is use of 1 bbl. of cement per cu. yd. of concrete in dam. Strength of concrete averages about 3,500 lb. at 28 days, approximately six times maximum stress of 600 lb. used in design and more than double specification requirements of 1,500 lb. Consideration of factors of impermeability and permanence account for maintenance of this cement proportion. Concrete placed within 3 or 4 ft. of the upstream face is enriched by increasing the cement content to 1.25 bbl. per cu. yd.—D. E. LARSON

The Osage hydro-electric project. G. R. STRANDBERG. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 243-248.—Recently completed project will serve as peak-load plant, serving St. Louis by being tied into the Keokuk hydro-plant and steam plants within St. Louis. Bagnell Dam, located 75 miles above confluence of Osage and Missouri rivers about half way between St. Louis and Kansas City, creates only large lake in Missouri and fourth largest artificial lake in United States, the length being 130 miles, capacity 1,200,000 acre feet (with a 30-ft. drawdown), area 95 square miles (61,000 acres), with drainage area above the dam 14,000 square miles. Average river flow is 10,500 c. f. s. fluctuating between 110,000 and 324 c. f. s. Bagnell Dam is of concrete gravity type 2543 ft. long and 148 ft. from bedrock to floor of 20 ft. wide bridge over dam. The water level can be raised about 105 ft. and a full year's flow can be equalized. Foundation rocks are dolomite and sandstone which lie 25 ft. or more below water section. Bottom widths are from 93 to 107 ft. The dam contains about 550,000 cu. yd. of concrete and 875,000 cu. yd. of earth and gravel were excavated. Capacity is initially 180,000 h. p. (6 units) with 2 service units and 2 additional main units of same type. The plant consisted of a 4-mile construction railway from Bagnell and 2 cubic-yard mixers located just downstream from construction bridge. Sand and gravel for concrete were dredged from river bed 3½ miles down stream from dam.—H. J. GILKEY

Construction of Ruskin power project. (See ENGINEERING DESIGN—DAMS.)

MISCELLANEOUS

When to use piling. A. C. EVERHAM. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 304-6.—The usual considerations controlling the use of piles are reviewed and comparisons made between concrete and wooden piles and between straight sided and tapered piles. Soil tests are mentioned and the reader is warned against drawing sweeping conclusions from small-area tests. For foundations under water, pre-cast piles 115 ft. long and 24 x 24 in. in cross-section were driven for a bridge at Newport News. These are longest piles ever driven and special precautions and methods were required to prevent deflection cracks and to secure proper alignment.—H. J. GILKEY

Glass and reinforced concrete (Glaseisenbeton) and its application for large concrete structures. H. CRAEMER. *Zement* (Germany), Dec. 11,

1930, V. 19, No. 50, p. 1185-8.—A new combination of reinforced concrete and glass, called "Rotalith," is described. New construction method used round glass bodies which are placed into forms and imbedded in concrete. Sides of glass bodies are grooved to secure firm contact between the two materials. Properties of glass as a building material are described. Compressive and tensile strength are very high, modulus of elasticity is also great; due to internal tensions glass is very brittle. A number of vault constructions are described and experiences mentioned. Fire tests showed that glass and reinforced concrete act as a uniform body and have high fire resistance.—A. E. BEITLICH

Construction plant on the Beauharnois canal job. *Contract Record Eng. Rev.*, Nov. 5, 1930, V. 44, No. 45, p. 1384.—In construction of Beauharnois Power Development in Quebec concrete at the power house site is being mixed in 2 two-yd. mixers with all ingredients proportioned by weight. Aggregate is supplied by primary and secondary rock crushers having capacities respectively of 250 and 75 tons per hr. and a sand machine of 65 tons per hr. Four traveling cranes are employed in pouring concrete.—G. M. WILLIAMS

Shaking and stirring of concrete. *Zement* (Germany), Dec. 25, 1930, V. 19, No. 52, p. 1238-41.—In discussion of a paper by Trèves and addresses delivered by Freyssinet and Séailles on importance of shaking and stirring of concrete in molds, two methods are described: (1) Vibrators act on outside of forms and cause packing of concrete mixture. Apparatus designed by Bérenguier, which is called "Vibropyl," is very efficient. (2) Pervibrators are always placed in interior of concrete mass and transmit their vibrations directly. Great advantages of both methods are described in detail. Uniformity of concrete and its compressive and tensile strength are considerably increased. Difficulties, which were encountered in practice and remedies for such cases are illustrated.—A. E. BEITLICH

Encasing wood stave pipe with reinforced concrete. *Contract Record Eng. Rev.*, Nov. 19, 1930, V. 44, No. 47, p. 1435.—To permit landscape improvement of a park through which runs a 13 ft. 6 in. wood stave pipe line of the Hydro Electric Power Commission of Ontario the wood pipe has been encased in reinforced concrete shell which will permit removal of the wood-stave material at some later date. Reinforcing consists of an inner and outer ring of circumferential bars designed to take all stresses due to water pressure. Longitudinal reinforcement is designed to care for maximum temperature changes to which the pipe may be subjected. To fully support the pipe during construction the invert was poured in short saddle lengths of 6 ft. 4 in., carrying the covering up the sides to 60 deg. above bottom center line of pipe. Construction joints were made watertight with keys coated with emulsified asphalt. Concrete (16,200 cu. yds.) was delivered by transit mixers and discharged into chutes leading to bell mouths from which it was directed into place by tubes.—G. M. WILLIAMS

Concrete construction control and practice in the United States of America. H. GRIESEL. *Zement* (Germany), Dec. 4, 1930, V. 19, No. 49, p. 1163-7.—Factors which influence strength and general properties of concrete are: (1) Physical properties, shape, surface and purity of sand. (2) Natural composition of aggregates, petrographic character and influence of gradation on compressive and tensile strength. (3) Necessary amount of water in mixture and consistency. (4) Mixing and transportation, effect of temperatures and weather. Construction of bridge over Hudson River between New York City and Fort Lee N. J. is described as an example of construction control in United States. Results of laboratory tests given illustrate relations between concrete strength and w/c, influence of gradation on strength and most economic mixture which gives desired plasticity and guarantees necessary compressive strength. Tables and curves show sieve analysis and compressive strengths obtained under different conditions.—A. E. BEITLICH

Construction methods—Bridge River tunnel. *Contract Record Eng. Rev.*, Nov. 12, 1930, V. 44, No. 46, p. 1425.—Power project of British Columbia Electric Railway Co. involves boring a 2½ mile tunnel through Mission Mountains. Completed tunnel is horse shoe shape equivalent to 14 ft. 3 in. circular section. Lining requires approximately 34,000 cu. yd. concrete. In pouring the lining the invert or lower portion of the horse shoe section is poured and shaped by template together with short standing walls at each side. On this concrete floor a track is placed to carry a "jumbo" car equipped with a concrete gun and hinged collapsible steel forms, which rest on the base section and after being toggled into position support themselves as well as the weight of the concrete after placing. A pipe equal in length to the section being poured is inserted between form and rock at its highest point, is attached to concrete gun equipment and as car moves backward and forward concrete is forced through and allowed to flow down laterally into position. Concrete is compacted by means of small air hammers striking the outer form surface.—G. M. WILLIAMS

New construction of a quay in Bordeaux—Bassens (France). EMIL STRASSBERG. *Bautechnik* (Germany), Sept. 26, 1930, V. 8, No. 42, p. 643-6.—Reinforced concrete quay, about 8200 ft. long, to replace old structure on wooden foundations, consists of reinforced concrete piers connected by reinforced concrete arches. Piers are about 15 ft. high, 18 ft. wide and each rests on 31 octagonal reinforced concrete piles 15.7 in. in diameter and 62 ft. long. Part of piles have inclination of 1:4, 1:8, or 1:10. Each one carries load of 27.5 tons. Piers are 22.6 ft. deep and reinforced with steel rails. Arches have span of 27.9 ft.; they are 33.5 in. thick at centers and 55.1 in. near piers. Longitudinal reinforcements are placed in lower part of arches and in top part above piers. There is no connection between them. Piers are partly protected by granite facings. Mixture consisted of 772 lb. cement, 14.2 cu. ft. fine gravel and 28.2 cu. ft. of coarse gravel and had a plastic consistency. Detailed description of arrangement of site of construction and piling work is given.—A. E. BEITLICH

Vibro system of concrete piles. *Contract Record Eng. Rev.*, Oct. 29, 1930, V. 44, No. 44, p. 1362.—Vibro system of concrete piling involves driving into the ground a plain steel tube with a cast iron conical shoe, placing reinforcement in the tube if desired, and filling the tube with concrete after which the tube is removed. Operations of driving and with-drawing tube are performed by same steam hammer which requires use of a comparatively light supporting frame. The supporting frames are built in sections and used in heights of 50 and 64 ft. for making piles 34 ft. and 45 ft. long. Cast shoe is placed in position on ground surface and hollow cylindrical shell or tube set in position and fitted to shoe to make a water-tight joint, after which it is driven to proper bearing. Hammer blow is transmitted to tube through a metal covered wooden driving cap which fits into a cast steel helmet which in turn is welded on to a steel casting which rests on the top of tube. If reinforcement is used, the hammer and helmet are raised and the fabricated metal lowered into position. Concrete is placed through an opening in the side of the steel helmet at such a rate that the cavity below ground level is full when the tube has been withdrawn 1/3 the distance. In withdrawing the tube after each upward stroke of the hammer the tube is permitted to descend about one half the distance which it is raised, thereby ramming and compacting the concrete and thoroughly filling the space occupied by the walls of the tube. This vibration tamping action results in the formation of corrugations on the pile surface which are claimed to increase bearing power. A two-ton, single acting steam hammer is used. In average ground the bearing power of 32 ft. Vibro pile is said to be about 150 tons.—G. M. WILLIAMS

Construction of Lacroze subway railroad system in Buenos Aires (Argentina). ALOIS POLLAK. *Zeitschr. Ostr. Ingenieur und Architekten*

Vereins (Austria), Nov. 21, 1930, V. 82, No. 47-48, p. 401-3.—Second subway line, built by Ferrocarril Terminal Central de Buenos Aires, connects north western part of city with center and serves as freight line during night hours. Total length of this 15 million dollar project is 22,000 ft., 12,400 ft. of which was built in open excavation. Tunnel, 1394 ft. long was constructed underneath Maldonado river. Latter was provided with reinforced concrete floor. Reinforced concrete predominated in entire construction. Total of 157,000 cu. yds. of concrete and 7275 tons of reinforcements were used. Inner width of single tunnel is 13.39 ft. and of twin tunnel 28.54 ft.; tunnel is 19.36 ft. high. Passenger stations are all of similar design, 426.5 ft. long and from 47.6 to 68.9 ft. wide. Ventilating shafts are placed every 426.5 ft.—A. E. BEITLICH

Chicago open-well method. W. J. NEWMAN. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 306-8.—The Chicago open-well method, devised by General William Sooy Smith about 40 years ago, is well adapted for penetrating blue clay of the Chicago region to hard limestone that lies as deep as 130 ft. below surface. Clay is stiff and can be excavated in 3 to 6 ft. sections and lagged with 2 or 3 in. beveled-edge lagging held by two metal rings on the inside. Excavation is by hand and circular buckets about 2 ft. high and 20 in. in diameter are hoisted with an electrically operated niggerhead winch. Concrete is placed by pouring directly into the well. The top 4 ft. of caisson is usually reinforced with hoops and vertical rods and mixtures commonly used have been 1:2:4 or 1:1:2 depending upon design requirements. The rings and lagging are often removed ahead of concreting. In blue clay the hole is usually dry but the method must be varied when water-bearing seams are encountered. Methods for meeting different aspects of water problem are outlined. Recently rotary excavation machines have been used satisfactorily for caisson work of Chicago type.—H. J. GILKEY

Underground construction. "Grundbau." M. BENZEL, 1930, B. G. Teubner, Leipzig and Berlin (Germany), fifth edition, 238 illustrations, R. M. 4.40. Reviewed in *Zeitschr. Ostr. Ingenieur und Architekten Vereins* (Austria), Dec. 5, 1930, V. 82, No. 49-50, p. 425.—First part deals with investigation of underground by means of test piles and drilling. Then excavating and different methods of pilings are described. Coffer dam and caisson jobs are illustrated. Third part shows protection of excavations against water and frost. Descriptions of modern tools and machinery are given in detail.—A. E. BEITLICH

Use of caissons in St. Louis sub-soil. S. W. BOWEN. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 309-311.—St. Louis bedrock consists of limestone at a usual depth of 50 to 80 ft. Open concrete caisson construction is quite successful in foundation work. A common outside diameter is 8 ft. with 9 in. walls although both vary. The shell is reinforced against both outward and inward pressure. Excavation may be accomplished with a dredging bucket, orange-peel bucket or by hand. No metal cutting edge is used except where boulders or other obstructions are found. Various loading devices are used for keeping the caisson properly aligned and outside lubrication with water jets is sometimes required. A moderate amount of shooting may also be employed near the base of the caisson. When quicksand or other water-bearing strata are encountered, pneumatic methods may be used by placing an air lock on top of the shell. After caisson has been properly grounded and rock cleaned the shell is filled with concrete somewhat leaner than that of which the shell is made. Buildings such as the Statler Hotel, Lennox Hotel and Civil Courts Building, are supported on foundations of this type.—H. J. GILKEY

Yearbook of the German Society of Engineering. "Jahrbuch der Deutschen Gesellschaft fur Bauingenieurwesen." S. BAER. 1930, V. D. I.-Verlag, Berlin (Germany), V. 5, 216 pp. R. M. 12.00. Reviewed in *Zeitschr. Ostr. Ingenieur und Architekten Vereins* (Austria), Dec. 19, 1930, V. 82, No. 51-52, p. 438.—Newest developments of construction methods are

described. Special consideration is given to construction of large storage houses, mining and marine structures. A list gives short descriptions of 310 great German and Austrian construction jobs completed in 1928 and 1929.—A. E. BEITLICH

Twelve month construction. JAMES S. TAYLOR. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 322-325.—The analysis includes a consideration of relative costs and other pertinent factors for the several types of construction. The usual technique and suggested precautions of preheating materials and use of salamanders are included for concrete. There is also a warning against fire risks and too rapid application of heat. Conclusion is that 12-month construction has proved its soundness and its right to united support. —H. J. GILKEY

Winter construction. "Mitteilungen ueber das Bauen im Winter." FRITZ EISEMANN. Kalk-Verlag G. m. b. H., Berlin W62 (Germany), R. M. 0.40. Reviewed in *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 64.—It is shown that winter construction can be made successful in many construction jobs by considering only few precautions and with only small extra costs. Small book treats subject from the economic viewpoint.—A. E. BEITLICH

Pre-cast concrete electrolytic tanks. HECTOR FARAH. *Canadian Eng.*, Dec. 9, 1930, V. 59, No. 24, p. 691.—Concrete has supplanted wood in the manufacture of 1216 pre-cast reinforced concrete tanks for the electrolytic refining of blister copper at plant of the Ontario Refining Co. Ltd., Copper Cliff, Ontario. The concrete cells, 11 ft. 3 in. long, 3 ft. 6 in. wide and 3 ft. 9½ in. deep, inside dimensions of precast members with wall thicknesses ranging from 3½ in. to 4½ in. were made of a special concrete mix consisting of 10 bags of cement per cu. yd. with very little sand and crushed rock not over ½". All steel reinforcement is protected by not less than 1 in. of concrete to prevent short circuits. Reinforcement consisted of specially designed mats of ¼-in. wire and spaced with 1-in. concrete blocks. Interior surfaces were protected by a lead lining and the exposed concrete painted with asphalt paint.—G. M. WILLIAMS

Chicago and North Western establishes quality control of concrete. O. F. DALSTROM. *Concrete*, Feb., 1931, V. 38, No. 2, p. 19-22.—In 1928 railroad adopted new specifications using water-cement ratio as basis of design for mixtures, following in general those adopted by American Railway Engineering Association. Results have shown necessity for being thoroughly organized and equipped for testing, inspecting and controlling concrete from time sources of supply are chosen until product is in forms, to insure proper interpretation of specifications and instructions. Because of great difference in new specifications from old, copy of specifications was sent to each contractor before bidding; special set of instructions on design of mix to division engineers, contractors and inspectors to accompany specifications; instructions to inspectors covering methods to be followed in field and requirements regarding water-control equipment on mixer. List of testing equipment and places of purchase was distributed to insure use of correct equipment. All field equipment was found without adequate water control in first year. This condition was completely corrected in 1930. Automatic time control devices are required. Test specimens of every mix are made in pairs, numbered in consecutive order throughout entire job, odd-numbered specimen being for 7-day test and following even-numbered specimen for 28-day test. Copies of test reports are sent to engineer of bridges, division engineer, inspector, and contractor, to guide and to stimulate interest. Samples of aggregates are submitted to railway company laboratory for test and approval. Railway's engineer visits each job occasionally or on request from division engineer or inspector, to settle questions regarding specifications or materials and equipment furnished by contractor.—C. BACHMANN

The manufacture and placing of large masses of concrete. L. BONNET. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Special precautions are necessary in mixing large quantities of concrete that have to be carried a great distance before placing in the work, to prevent segregation. Moreover manufacture of concrete in large quantities demands perfected plant both for actual mixing and for proportioning. During extension work in Port of Antwerp interesting observations were made in regard to concrete made from Rhine gravel and sand. Tests showed that best proportions of sand and gravel to use in making large masses of concrete are as follows: (1) 10 parts gravel to 7 sand, when the size of the gravel does not exceed 1.18 in. and when lead is short. (2) 10 parts gravel to 7½ sand, when the size of the gravel does not exceed 1.18 in. but when lead is long. The same proportion can be used if gravel is coarse while lead is short. (3) 10 parts gravel to 8 sand when gravel is coarse and lead is long. The second part of paper describes installation for making concrete in extension works of Port of Antwerp.—EDITIONS LA TECHNIQUE DES TRAVAUX

Handbook for reinforced concrete construction. "Hilfsbuch fur den Eisenbetonbau." VICTOR HIETZGERN AND ARNOLD ILKOW. 1930, Julius Springer, Vienna (Austria), R. M. 5.80. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Book is treatise of entire field of structural materials and deals with proper preparation and working in reinforced concrete construction. Book is devoted to use by contractor and designer.—A. E. BEITLICH

Economy of distribution towers for cast concrete in superstructure construction. "Die Wirtschaftlichkeit des Gussbeton-Foerderturmes im Hochbau." R. FINNERN. 1930, W. Kohlhammer, Stuttgart (Germany), R. M. 3.00. Reviewed in *Zement* (Germany), Nov. 27, 1930, V. 19, No. 48, p. 1146.—After showing historical development of concreting methods, author describes economical properties of distribution arrangements for cast concrete. Conditions are illustrated under which installation of towers is advisable. Different examples are given and discussed from an economic view point.—A. E. BEITLICH

Twelve-month construction. JAMES S. TAYLOR. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 322-325.—During last decade winter construction has greatly increased as compared with period before the World War. Some of the causes underlying the change are: (1) pre-war experience with winter construction was greatly multiplied by war-time undertakings, (2) twelve-month construction of many types has a sound economic foundation, (3) the problem affects so large a section of the community that it is of importance to many other lines of business as well as to the public generally. The process employed in the winter construction of concrete consists usually of heating the fine and coarse aggregates, heating the mixing water, protecting stock piles, and housing materials affected by moisture, as well as entire mixer and plant. The temperature of concrete when placed should be above 60°. Protecting concrete while placing and curing is more important in building operations than in bulk work such as bridge and dam construction. Relatively smaller section of a beam or column is more readily and seriously affected by freezing than larger mass of bridge abutment or section of dam. Bulk concrete also provides more heat in setting. It can be housed very cheaply because of its compactness. The housing of building construction is more expensive, but it is seldom necessary to house more than story under construction. The heating is usually done with salamanders.—D. E. LARSON

Use of caissons in St. Louis sub-soil. S. W. BOWDEN. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 309-311.—In order to meet unusual foundation conditions existing in St. Louis, there has been developed, in recent years, a rein-

forced concrete caisson which can be sunk to rock by open-well method, using air locks only where quicksand and water are encountered in such quantities as to make its use absolutely necessary. These caissons are hollow cylinders whose inside diameter is usually determined by the size of bucket used in excavating. Walls are made of such thickness as to give the proper outside diameter for the load to be supported. An outside diameter of 8 ft. is frequently used with 9-in. walls. For smaller sizes, excavation is by hand or by small orange peel bucket. No metal cutting edges are used except where boulders or other obstructions are apt to be encountered. Reinforcement consists of sufficient metal, both vertical and horizontal, to prevent cracking of walls, and this is placed in both faces. The minimum reinforcement generally used consists of bars $\frac{5}{8}$ in. in diameter, 12 in. from center to center for the hoops, and 24 in. for the verticals. Where it is likely that air will be used, the reinforcement is designed to take the unbalanced internal pressure, using unit stress of 15,000 lb. per sq. in. for the steel.—D. E. LARSON

Construction joints in concrete. NORMAN DAVEY. Dept. of Scientific and Ind. Research Build. Research Station (England), *Special Report No. 16*, 74 p.—The disadvantages of improperly made construction joints are mainly lack of strength and adhesion at the joint, efflorescence at joints, disintegration due to freezing of water in joints, susceptibility to percolation of moisture, disintegration due to penetration of substances in solution and failure under action of heat. Laitance at joint which is detrimental to proper bonding is usually due to excessive mixing water or even excessive ramming causing segregation of aggregate. Scum, dirt, clay, oil or grease should also be removed. Consistency of concrete should be carefully controlled so that mix will flow sluggishly when tamped into position. Method of washing joints with soda has been tested and found to be in no way harmful, provided all trace of soda is removed. Chipping and wire brushing old concrete has given good results but age of old concrete appears to be of considerable importance. Portland cement may be weakened by chipping while concrete is still green (at 24 hrs.), while after concrete has thoroughly hardened there is a falling off of bond strength due to difficulty of chipping and greater differential shrinkage between new and old concrete. Aluminous cement gives lower bond strength at earlier ages owing to greater shrinkage. For rapid hardening cement, chipping or treating the surface with hydrochloric acid and in both cases grouting with neat cement gives 70 per cent efficiency while for aluminous cement, acid treatment gives better efficiency. Results of tests on joints with old concrete prepared with proprietary compounds are given, and in most cases good results are registered. For applying new concrete to prepared surface of old, a sufficiently plastic mix must be employed to allow it to enter all interstices. A slump of 2 in. is recommended as minimum, while too wet mixes should be avoided. Durability of joints appears to increase with age. For two-layer work upper courses should be laid on lower before it has appreciably hardened. It is found that high adhesive strengths with a joint give watertightness. Aluminous cement concrete does not adhere so readily to rapid hardening types of cement as to ordinary portland cement. Joints in foundation rafts and footings should be cast to their full depth in one operation if possible. Any joints formed should be normal to the compressive stresses. Columns should be cast up to point just below the soffit of beams and left to set before beam and top of column are poured together. Beams may be stopped best at middle thirds, joints at points of excessive shear being avoided. Slabs should not have a horizontal joint and are best stopped at middle thirds.—JOHN E. ADAMS

Progress in reinforced concrete super structure construction during the year 1929. WILHELM PETRY. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 142-92.—Important changes were made in preparation of sand and gravel for concrete by producing a greater number of gradings and using better proportioned mixtures, which fact resulted in

considerable increase in compressive strength. Many valuable experiences were obtained in winter construction methods during severe cold weather periods of winter 1928-29. Modern construction methods and improved equipment make winter jobs economic. Great progress was made in construction with sliding forms; system Heine was invented and used for erection of two monolithic chimneys (459.4 ft. high with 20 ft. inner diameter at top), built for power station in Schwandorf (Germany) (cf. O. Hoffmann, *Zement* (Germany), Aug. 28, 1930, V. 19, No. 35, p. 828-34) in a period of only 3½ months. About 8.2 ft. per day were concreted. Six silos were built for Wicking cement plant in Neuwied (Germany), which are 65.6 ft. high and 26.3 ft. in diameter; walls are 7.9 in. thick. System Rohmeyer was used for construction of grain silo in Duesseldorf (Germany), which consists of 10 circular cells with diameters of 26.9 ft. Complete winter structures are 4 cement silos with 8800 tons capacity in Blaubeuren (Germany). Work was started in Nov., 1929, concrete was placed daily and work was completed in 5 weeks, with interesting construction details. Torkret method for concrete roofs saves much time and labor. Arrangement of site, transportation of materials and use of newest machinery made construction of office building Sprinkenhof in Hamburg (Germany) a full success. Two towers distributed about 235 cu. yards of concrete (in 10 hr.) daily. A new type of concrete pump was used with great advantage for number of structures (cf. W. Haevernick, *Bauingenieur* (Germany), Feb., 1930, V. 11, No. 7, p. 100-2; *J. Amer. Conc. Inst.*, Sep., 1930, V. 2, No. 1, Abstr. Sect., p. 18; Heidorn, *Bauingenieur* (Germany), May 30, 1930, V. 11, No. 22, p. 381-4). In several office buildings, skeleton structures, engineering efficiency gave excellent results. Of special interest are construction of City Hall in Ruestringen (H. Kuball, *Der Bauingenieur* (Germany), May 16, 1930, V. 11, No. 20, p. 349-52), buildings of Englehardt brewery (Herbst, *Zement* (Germany), Aug., 14, 21, 1930, V. 19, No. 33-34, p. 776-9, 800-4, *J. Amer. Concr. Inst.*, Nov. 1930, V. 2, No. 3, Abstr. Sect. p. 78) and warehouses of Merchants Corporation in Barmen (*Zement* (Germany), Oct. 30, 1930, V. 19, No. 44, p. 1039-41). Industrial structures, built during 1929, include several large factories, storage houses of nitric acid plant and coal house of a mine. Results of tests of Austrian Government concerning use of high quality steel for reinforcements of concrete columns are included.—A. E. BEITLICH

Special carfloat used to distribute bulk cement. (See MATERIALS—CEMENT.)

Structural design and construction of concrete improved in 1930. (See ENGINEERING DESIGN—MISCELLANEOUS.)

ROADS AND PAVEMENTS

Concrete road junctions. R. A. B. SMITH. *Concrete Constr. Eng.* (England), Jan. 1931, V. 26, No. 1, p. 35-41.—In description of construction of concrete road junctions, order in placing concrete for parts of junction is outlined and illustrated for different types of junctions. Diagrams show different methods of placing construction joints. The simple case of one road meeting another at right angles is first considered. For two roads intersecting at right angles six arrangements of joints are given. For three roads meeting in which the roads have different widths a special arrangement for junction is given.—JOSEPH MARIN

Nation constructs 10,200 miles of concrete roads in 1930. *Concrete*, Jan., 1931, V. 38, No. 1, p. 13-14.—With December figures partly estimated, total yardage of concrete pavements, including roads, streets and alleys, should be about 144,000,000 sq. yd. in 1930, or about 3 per cent over 1929. Trend toward greater use of high grade equipment is seen in fact that 32 states

now require machine finishing of concrete pavements, compared with 26 states a year ago. Sixteen states now demand definite 28-day strength, compared with 13 states a year ago. Definite water-cement ratio is specified by 24 states, compared with 17 a year ago; 29 specify a minimum quantity of cement per cu. yd. of concrete. All states now specify a minimum mixing time, ranging from 50 seconds to 1¾ minutes. All except two require attachment of timing device to control time of mixing, and all except one require measurement of mixing water. Cross-sections of concrete highways are now required to have some type of thickened edges in all but seven states.—C. BACHMANN

SHOP MANUFACTURE

Testing of artificial stone, especially concrete block. R. AMMON. *Chem. Techn. Rundschau* (Germany), 1930, V. 45, p. 157-9.—Experiments were made to determine weight per cu. ft., specific gravity, porosity, absorption properties, hardness, durability and resistance to abrasion of several types of artificial stone and concrete block.—A. E. BEITLICH

Recent noteworthy executions in cast stone. *Betonwerk* (Germany), Aug. 30, 1930, V. 18, No. 35, p. 491.—A description of recent noteworthy executions in cast stone includes that of a large monument, several pieces of garden sculpture, tombstones and a small building illustrated. Each item of execution is analyzed in detail, in a manner to indicate the kind of cast stone, how made and finished. The name of designer in each case is indicated.—HERMAN FRAUENFELDER

Profitable manufacture of concrete building units. FRED A. SAGER. *Concrete*, Feb., 1931, V. 38, No. 2, p. 27-31.—In working out uniform production rate and scheduling output, year's business is analyzed. Based on estimates of use, stock for new 2 by 4 by 8-in. brick was arbitrarily set at 30,000 units for first year and included with the five major items of production of previous year. Total stock Jan. 1, was 70,000 equivalent 8-in. units, made up of quantities as determined from table given for five major items and 30,000 brick. Plan is to increase production to 30,000 units per month for most of the year, allowing for severe cold weather. Estimate of future year's business is on data of three or more past years, when records of years considered are shown by curves, continuation of which in next year serves as basis for that year's production. As year progresses, actual deliveries running ahead or behind estimated deliveries will be indicated and program altered accordingly. Items to be produced in any month will be determined by knowledge of business ready for delivery or in immediate prospect. For ready reference, records of manufacture, sales, and stocks are desirable in the form of diagrams or graphs, which can be projected into the future.—C. BACHMANN

Manufacturing and selling problems occupy products industry. *Concrete*, Feb., 1931, V. 38, No. 2, p. 13-15.—Continued use of concrete masonry and back-up units in large buildings, sustained interest in light-weight aggregates, development of use of colors, and growing attention to manufacturing and selling problems are some of outstanding developments in concrete products industry of 1930. Causes of shrinkage cracks in masonry walls are definitely determined. Cracking can be avoided if units are completely cured and air-dry before being used. Research associate at Bureau of Standards will inaugurate research program on cinder concrete building units covering a number of years. Progress has been made in New York with development of light-weight aggregate by sintering ashes and cinders from low-pressure boilers and other sources from which cinders, in raw state, are not suitable as aggregates. Improved design has made light-weight unit possible in some

cases where ordinary aggregate are used. Vibrating equipment is being used, especially in cast stone, concrete pressure pipe and drain tile manufacture. Use of color in cast stone and ordinary products is receiving increased attention and manufacture of concrete burial vaults is in more satisfactory condition. In new construction method, concrete masonry units, laid up as dry masonry, are "welded" by cement gun which shoots mortar into joints and coats surface. New specialty is metal-faced light-weight concrete slab or block for exterior or interior trim using any commercial metal as facing material.—C. BACHMANN

Spun concrete and reinforced concrete pipes., ERWIN MARQUARDT. *Bautechnik* (Germany), Sept. 16, 1930, V. 8, No. 40 p. 587-602.—After discussing historical development author explains fundamental principles of spinning process. Movement of materials in mold and necessary speed of rotation are illustrated by a number of mathematical equations. Some separation of aggregates and cement is always caused by centrifugal force, coarse parts being accumulated in outer portion of wall while lighter parts and cement are forced to interior. Following means for prevention of separation are advised: (1) Mix should not be too wet; plastic concrete has best consistency. (2) Aggregates should have rough surface and compact form, (3) Aggregates should be of as similar petrographic character as possible. Gradings over 0.6 in. should be avoided. (4) Manufacturing of pipes can be carried out in different layers with adjusted speed of rotation. Different manufacturing processes now in practice in United States and Germany are: (1) Dyckerhoff and Widman in Dresden (Germany) make pipes about 50 ft. long with inner diameter from 3.8 to 5 ft. They are reinforced with cylindrical wire cages with one or two spiral windings. Pipes are suitable for pressure lines under low heads. Belljoints connect pipes with small diameter while spun precast collars, reinforced with 2 spirals are used for larger sizes. (2) A high quality pipe for high pressures which is absolutely watertight is a steel sheet pipe covered externally and internally with layers of dense concrete. Spiral windings are wound first on special machine and then tightly placed inside and outside of steel cylinder. Outer coating is made of high early strength cement and placed in standing form while inner layer is spun after mounting steel form on driving wheels. Mixtures range from 1:3½ to 1:4. After spinning is completed (about 20 min.) excess water which is squeezed out is removed. Pipes are taken out of forms after 24 hours and stored in yard being kept wet by sprinkling for several days. Transportation, placing and jointing are described. (3) Hume process widely used in Europe and United States, is divided into 2 distinct periods. Forms are rotated first at slow rate of speed (200 to 300 r. p. m.) which forms a dense layer of uniform thickness in wall of pipe. Excess water is drained and interior surface is smoothened with a round steel rod during second spinning period (550 to 700 r. p. m.). Forms and pipes are carried to steam kilns and cured from 8 to 12 hr. with low temperature steam. Further storage is in yard. Joints are made with collars and hemp packings. Inside joints and one outside joint between collar and pipe are made in field. (3) Pipes of Vianini type are usually made in France. Molds do not rotate on wheels but are fastened between 2 side discs which movement is claimed to be free from vibrations. Different types of joints are illustrated. (5) Lock Joint Pipe Co., Ampere, N. J., manufactures pipe from 1.3 to 9 ft. in diameter for high heads. Longitudinal reinforcing rods are tied to wire cages by hand or by welding. Ingenious design of joints makes placing and jointing very easy. (6) Main feature of system by Moir and Buchanan is special feeder which at the same time secures uniformity of wall thickness. Pipes are of very dense structure. (7) Short description is given of several asphalt coated pipes. Last part discusses properties of spun concrete pipes such as strength, static tensions, water impermeability, resistance against mechanical abrasion and chemical influences, hydraulic capacities and economy. —A. E. BEITLICH

Materials

Aggregates

Concrete from Slag. J. Bentkowsky. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—The report deals with a series of tests carried out by the Institute of Metals in Leningrad with a view to studying possibility of using a given slag for manufacture of concrete, and determining the mix from these tests in relation to workability and strength.—Editors N. I. A. T. M.

Influence of split on strength of cast concrete. Luz David. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 32-7.—Field tests were made to study improving effect of additions of split to concrete on its compressive strength. One high-grade quartzite split and a low-grade quartz porphyry split were used in 1:1:5 and 1:2:4 (cement-split-sand) mixtures. It was found that compressive strength is considerably increased by even small additions (1/5 of sand) of split. This increase is greater when high early strength cement is used. In this latter case, low-grade split seems to give greatest increases in strength. Question whether such additions are economic in practice or not is discussed and ideal conditions described. Price of split and facilities at site of construction are chief governing factors.—A. E. Beitlich

Plant design and practices—Part 1. G. W. Maisak. *Rock Products*, Feb. 28, 1931, V. 34, No. 5, p. 71.—The first problem considered is that of a feed hopper to a primary crusher, in this case a jaw crusher. A common form uses old rails for lining. These should never be set at an angle greater than 40 deg. to minimize damage from surges of rock. This type of hopper is affected by weather conditions and the amount of dirt coming from quarry. Stone-on-stone type of feed box is favored, as once built it needs no further attention. In the larger plants it should be made of fabricated steel.—Edmund Shaw

Barges bring aggregate to ready-mixed concrete plant at Tacoma. Chas. F. A. Mann. *Pit and Quarry*, March 11, 1931, V. 21, No. 12, p. 56-58.—At new concrete plant of George Scofield Co., which produces Tru-Mix concrete, aggregates are received on barges, chiefly from pits belonging to an interlocking company. Barges are unloaded by hammerhead crane built in company's shops and which rests on a steel framework above piling. The 65-ft. boom carries a 2½-cu. yd. clamshell bucket, and handles 65 cu. yd. per hour into a 10-cu. yd. steel hopper. Material drops to belt conveyor which delivers to conveyor on upward slope of 15 deg. to the storage house. Material can be deposited on bunker-storage belt or carried to bunkers above mixing platform. Bunker-storage belt delivers to the bins. Beneath the storage a conveyor will reclaim the material and beneath the bunkers of the mixing plant the steel chutes have cut-off gates to the scale pan above the mixer. Pan has steel chute to 2-cu. yd. mixer beneath. Cement now measured in a small steel dump-cart will be handled same way as aggregates. A mixometer provides an accurate check on concrete inside the mixer. Concrete is delivered by trucks.—A. J. Hoskin

Aerial photography in the aggregates industry. Howard Maxwell. *Rock Products*, Feb. 28, 1931, V. 34, No. 5, p. 41.—It is often imperative to have maps of aggregate operations, construction projects and sites for either of these. Aerial photographs make best maps, being absolutely accurate and showing details that could not be shown in maps without excessive cost. Now even amateurs who thoroughly understand photography may make aerial photographs that will serve ordinary purposes. Comparatively small areas can be covered with one or two exposures. An exposure made at 7,200 ft. with

a camera of ordinary size will give an accurate map of approximately 200 acres at a scale of 1 in. to 600 ft. This can be enlarged to any scale up to 1 in. to 200 ft. Wide angle lenses are not recommended as they are liable to produce distortion and are usually slower than other lenses.—EDMUND SHAW

Fifth annual report of the engineering and research division. STANTON WALKER. *Nat. Sand Gravel Bul.*, Feb., 1931, V. 12, No. 2, p. 29-37.—Brief summary of work under way in laboratory of National Sand and Gravel Association includes: (1) Effect of characteristics other than grading on concrete-making properties of concrete aggregates—includes studies of mineral composition, surface texture, strength, shape and miscellaneous physical properties of coarse aggregate particles. (2) Effect of size and grading of aggregate on quality of concrete—deals principally with coarse aggregates, although preliminary studies of fine aggregates have been made. (3) Method of proportioning concrete—the development of a logical and scientific basis of proportioning concrete is essential to studies of aggregates for concrete, and is particularly pertinent to the investigations mentioned above. (4) Tests of sand—studies to date have been of a preliminary and exploratory nature, but they indicate the need for a comprehensive investigation. (5) Characteristics of gravel ballast—studies of physical properties and service records of gravel ballast and preliminary work in a study of methods for testing stability of ballast of different types and gradings. The principal efforts of the laboratory for the coming year will be directed to the study of the following problems: Effect of coarse aggregate characteristics other than grading on strength of concrete; methods of tests for durability of aggregates and concrete; correlation of tests of sands with their concrete-making properties; and studies of size and grading of aggregates with particular reference to methods for proportioning fine and coarse sizes for concrete.—P. MCKIM

Produces aggregates and central-mixed concrete for Philadelphia area. *Pit and Quarry*, Feb. 25, 1931, V. 21, No. 11, p. 27-31.—Aggregates used by Liberty Corp., are dredged from the Delaware River and transported in barges to dock at plant. Material averages 60 per cent sand, 40 per cent gravel. Dredge is of steel construction and of the bucket-ladder type. Products of four grades are chuted to barges. At the unloading dock materials are handled by a gasoline crawler crane and electric locomotive crane with clamshell buckets. First crane dumps oversize stone to hopper of crushing-screening department, and electric crane unloads sand and gravel to truck-loading bins or distributing-plant hopper. All bins have steam coils to heat aggregates in freezing weather. The dry-batching unit has a 2-compartment steel bin of 200 ton capacity. Aggregates are weighed in a 2-compartment 6-ton batcher with automatic attachment, and are discharged directly into trucks. Mixer unit has a 4-compartment steel bin of 300 ton capacity for aggregates in three compartments, and four carloads of cement in the fourth compartment. Bulk cement is power-scooped from cars and elevated to storage bin. Cement is withdrawn from its bin by a screw conveyor to the cement weighing batcher. Batches are weighed into a 3,000-lb. weigh hopper and then discharged to 3-cu. yd. mixer. Deliveries of concrete are made by a fleet of trucks with 4-cu. yd. agitator bodies.—A. J. HOSKIN

Tests of plain and reinforced Haydite concrete. F. E. RICHART AND V. P. JENSEN. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 674-693.—This paper furnishes data on structural properties of poured Haydite concrete. The mixtures include concretes made with fine and coarse Haydite aggregates, natural sand and coarse Haydite, and for comparison, sand and gravel or broken limestone. An initial series of tests was made on beams and cylinders of machine-mixed concrete to secure information on mixes suitable for ordinary reinforced-concrete construction, with particular attention to securing workability, surface finish, and freedom from segregation. The investigation in-

cluded tests on 75 beams, 27 pull-out specimens, and nearly one thousand 6 by 12-in. cylinders. With the consistencies used, no difference was noted in work required to place concrete, or in appearance of the molded surfaces of beams made with gravel, limestone, or Haydite. The gradation and texture of Haydite aggregates required a somewhat greater water-cement ratio than gravel aggregate in similar mixes, and consequently gave a lower strength concrete. Absorption allowance is not fixed, but depends on initial moisture content of aggregate as used. For equal water-cement ratios, strengths of concretes made with sand and gravel, sand and coarse Haydite, and fine and coarse Haydite are very nearly equal. There was no indication that strength was limited by strength of aggregate. The modulus of elasticity of Haydite concrete is markedly lower than that of ordinary concrete of like proportions. The density ranged from 93 to 126 lb. per cu. ft. depending upon the amount of sand present, variation in mixture, and moisture control of aggregate. Up to 6 mo. at least, increase in strength with moist curing is comparable with gravel concrete. Bond and diagonal tension resistance are about the same proportion of compressive strength for Haydite concrete as for gravel concrete.—GLENN MURPHY

CEMENT

About the use of pumice as a raw material for cement manufacture. M. STILLER. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 51-4.—Possibilities for use of pumice as a raw material (instead of clay) are discussed and investigated by synthetic burns with a number of different kinds of pumice and trass. Clinkers were burned in experimental shaft kiln. Analysis of raw and finished products are given. Clinkers show flash sets and need additions of gypsum from 4 to 6 per cent to obtain normal setting properties. Alkali content is high, which fact lowers burning temperature considerably. In a rotary kiln a great part of alkali can be volatilized. Economic importance of these methods is illustrated.—A. E. BEITLICH

Quicker tests of cement and concrete. H. KREUGER. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Several preliminary tests have been made in order to investigate the possibility of deciding quickly (in 1 or 2 days instead of 28 days) if a cement is fit for delivery. Tests have proved that there probably may be a fairly good concordance between the normal crushing strength of cubes by 28 days combined air-and-water curing and by steam-hardening in a few hours. Method may be of some use also by quicker testing of concrete. It, however, is not yet satisfactorily developed, but seems worthy of some attention.—EDITORS N. I. A. T. M.

Portland cement plant in Sonora, Mex., to start production in March. *Pit and Quarry*, Feb. 25, 1931, V. 21, No. 11, p. 20-22.—Wet-process plant of Cemento Portland Nacional, S. A., at Hermosillo, Sonora, is nearly completed with daily capacity of 500 bbl. Abundance of fine limestone and clay occur close to plant will be hauled by motor trucks. Stone will pass through gyratory crushers and will then be conveyed to the mill-feed bin or stored. Raw grinding will be 2-compartment mill, the slurry being pumped to four tanks having pneumatic agitation. Pumps will transfer slurry from the tanks to the kiln feeder. Clinker will be taken by drag conveyor for storage or delivery to finish compartment mill. Gypsum is fed by the same conveyor. Cement will be handled to storage silos by a screw conveyor and a bucket elevator. From the silos, cement will be withdrawn to elevator to packer bin for 2-tube packer.—A. J. HOSKIN

The testing of cements by means of earth-moist and plastic mortars. J. P. LOEVENTHAL. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Former comparative investigations carried out by Danish state testing

house are noted, in which Swiss prism method and common hammer method were used; in continuation, a series of tests dealing with mortars of portland cement, aluminous cement and a Danish pozzuolana cement (Moler cement) is discussed. Test-pieces were made in compact moulds and were of consistencies ranging from earth-moist to fluid. They were tested at 7 and 28 days. It was shown that at both dates strength curves of portland and Moler cements ran fairly similarly, although there was no proportionality. In contrast with this, mortars of aluminous cement, of low water content, showed an increase in strength between 7 and 28 days, while mortars of high water content a fall in strength. The various methods of testing are discussed.—EDITORS N. I. A. T. M.

Chemical action of gypsum and other retarders on portland cement clinker. L. FORSEN. *Zement* (Germany), Nov. 27, Dec. 4, 11, 1930, V. 19, No. 48-50, p. 1130-4, 1155-60, 1184.—Reactions which take place when cement and water are mixed and setting problem of portland cement were studied. Especially effect of different retarders was investigated and explained from standpoint of chemistry of complex compounds. It was found, that chiefly calcium aluminates react with retarding agents. Clinker and cement were agitated with water and amount of alumina, which went into solution was determined by chemical analysis. Isolation of several compounds was possible and these were studied under microscope. Retarding effect of following soluble calcium salts was studied: $CaSO_4+\frac{1}{2}H_2O$, $CaSO_2+2H_2O$, $CaCl_2$, $CaBr_2$, CaI_2, $Ca(ClO_3)_2$, $Ca(IO_3)_2$, $Ca(NO_3)_2$ and $Ca(NO_2)_2$. A number of structural formulas for products which are formed in such reactions are derived. Behavior of mixtures of portland cement and high alumina cements are discussed.—A. E. BEITLICH

The strengths of Swiss portland cement—Die festigkeiten der schweizerischen portlandzemente. M. Ros. Eidg. Materialprüfungsanstalt, Zurich. *Wochenschriift des Niedersteörreischen Gewerbevereines* (Switzerland), April 17, 1930, 4 p.—Greatadvances have been made in the improvement of the strength qualities of cement in Switzerland during the last 50 years. Average tensile and compressive strength on moist rammed 1:3 test mortars increased approximately 4 fold between 1881-3 and 1925. Curves show these increases in strength. High early strength cements are included for recent years. Graphs also show tensile and compressive strengths as well as flexure (bending) at 3, 7 and 28 days. Comparison of compressive and tensile strengths for moist rammed and plastic 1:3 test mortars show that the compressive strengths of the plastic are about 60 per cent of those of the moist rammed at the same ages, whereas, the tensile strengths of the plastic mortars are about 30 per cent greater than those of the moist rammed.—H. G. FISK

Preparation and characterization of dicalcium silicate, tricalcium silicate and tricalcium aluminate. I. WEYER. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 48-51.—In connection with work on reactions between kaolin and lime, it was necessary to prepare chemically pure the following compounds: dicalcium silicate, tricalcium silicate and tricalcium aluminate. (1) Dicalcium silicate was prepared from calcium carbonate and silica in a 2:1 mixture by alternating heating, at 2552° F., cooling and mixing. The two modifications, β- and δ- form were separated by fractionated centrifuging in a methylen iodide-acetylen tetrabromide mixture. Optical constants were investigated by microscope and X-ray analysis. Results obtained are in close agreement with findings of American investigators. (2) Tricalcium silicate was prepared at 3452° F. and was obtained in very pure form after several heat treatments. Free lime was present only in small traces. Optical properties are given which prove again unobjectionable existence of this compound. X-ray determinations were made. (3) In a similar manner tricalcium-aluminate was obtained in well crystallized form. Three tables show lines obtained by

X-ray analysis with relative intensity, and they are compared with results by Brownmiller and Harrington. Sixteen references are given.—A. E. BEITLICH

Small-piece testing for strength of cement mortars—Part 3. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), Feb., 1930, V. 33, Supplemental Binding, No. 2, p. 46-7B.—Formula is given for relation between compressive strength of 1:3 cement-sand mortar tested by ordinary test method and compressive strength tested with small cylinder method. Effect of water-cement ratio on compressive strength measured by small-piece testing method is illustrated and discussed for several different cements. Constants of strength formula (derived in previous paper) were calculated and difference pointed out between portland cement, blast furnace slag cement, soliditit, neo-soliditit and high alumina cements. Subject is still under investigation. (cf. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), March, 1930, V. 33, Supplemental Binding No. 3, p. 290-5B; JOURNAL A. C. I., Dec., 1930, V. 2, No. 4, *Abstracts* Sect., p. 99.)—A. E. BEITLICH

Zeolites and cements. HENRI LAFUMA. *Revue materiaux constr. trav. publics*, Jan., Feb., 1931, No. 256, 257, p. 1-5, 45-49.—The first part of this paper presents a study of the zeolites and the hydrated salts of cement. Hydrated aluminates and corresponding double salts occur uniformly as hexagonal crystals, negatively monoaxial. Greatly marked symmetry of these crystals—for such complex substances—and their resemblance to similarly hexagonal crystals of hydrated lime, lead to conclusion that their crystalline structure is determined by only a part of the molecules, namely molecules of hydrated lime (a common constituent). In this way are derived all properties so closely resembling salts of zeolites. This resemblance in structure also results in possibility of numerous solid solutions between all these bodies and furnishes an explanation for widely divergent results obtained by different investigators. If similar properties were to be discovered for silicates of hydrated lime, these crystalline structures would assume an important role in determining hydraulic properties. It would be interesting to establish hydraulicity as a specific property of hydrated salts possessing certain structures. Second part of paper reviews above statement, though limited by scant available knowledge concerning silicates of hydrated lime. A new concept, "the adherence structure," arises to supply a novel interpretation of "hydraulicity" phenomenon. Reference is made to original tests and to researches in different countries.—M. A. CORBIN

Studies on calcium ferrites and iron cements—Part 1. SHOICHIRO NAGAI AND KATSUHIKO ASAOKA. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), April, 1930, V. 33, Supplemental Binding, No. 4, p. 130-3B.—In systematic study of combination between ferric oxide and lime in high lime mixtures such as $3CaO:1Fe_2O_3$, $5CaO:1Fe_2O_3$, etc., determined ferrite with highest possible lime content was determined as $2CaO.Fe_2O_4$ and not $3CaO.Fe_2O_3$. Mixture of $3CaO:1Fe_2O_3$ was heated at various temperatures above 1652° F. and properties of final products, especially their solubility in 0.5 N-HCl, were studied. Dicalcium ferrite $2CaO.Fe_2O_3$ dissolves easily while $CaO.Fe_2O_3$ remains insoluble. Similar results were obtained with mixtures of $5CaO:1Fe_2O_3$ and $2CaO:1Fe_2O_3$. Dicalcium ferrite is easily prepared by heating mixture above 2192° F. It was not possible to prepare $3CaO.Fe_2O_3$ even at temperatures between 2550° and 2640° F. Results of heating tests of mixtures: $3CaO:Fe_2O_3$, $5CaO:Fe_2O_3$ and $2CaO:Fe_2O_3$ are given in tables. (cf. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), May, June, July, Aug., 1930, V. 33, Supplemental Binding, No. 5-8, p. 161-4B, 190-2B, 256-9B, 312-5B; JOURNAL A. C. I., Dec., 1930, March, 1931, V. 2, No. 4-7, *Abstracts* Sect., p. 96, 190.)—A. E. BEITLICH

A study of fourteen brands of standard portland cements and four early-strength cements. C. H. SCHOLER AND L. H. KOENITZER. *Proc.*

A. S. T. M., 1930, Part 2, V. 30, p. 581-593.—Only two or three of brands of cement reported in this paper were included in the 1928 report of Committee C-1 of the A. S. T. M. Data is presented on the relationship between chemical ingredients of each brand of cement, time of set, normal consistency, fineness and specific gravity of the cements, slump and flow tests, density of mix, tensile and compressive strengths of mortars, and flexural and compressive strengths of concretes. The general impression from the data is that of rather unusual uniformity. The principal variation in the different brands was their sensitivity to a small change in water content in the mixes at basic water content and above.—GLENN MURPHY

Canada Cement adapts plant at Hull to wet process of manufacture. W. E. TRAUFFER. *Pit and Quarry*, Jan. 14, 1931, V. 21, No. 8, p. 27-32, 26 ill.—Two years ago Canada Cement Co., began conversion of its No. 3 plant at Hull, Que., from dry to wet process. Stone from quarry is loaded by electric shovel into side-dump cars, hauled to plant by gasoline locomotive, hauled two carloads at a time, up an incline and dumped into jaw crusher. From crusher, stone is fed to hammermill discharging onto belt conveyor and is carried to the 2,200-ton storage. Clay is loaded by a gasoline-electric shovel, hauled by gasoline locomotive and dumped through a grizzly into wash mill. The slip is pumped about 1,800 ft. to basin with traveling agitator. Stone passes to feed bins above two raw-grinding mills, each mill having a 600-h. p. motor. Slurry, with 40 per cent water content, goes to correction basins with mechanical and pneumatic agitators, discharging to mixing basin with a quadruple mechanical and compressed-air agitators, then into storage basin with traveling agitator. Slurry is pumped thence to ferris-wheel feeder of 365-ft. kiln, 11 ft. 3 in. and 10 ft. in diam. A 10 section, single-row revolving cooler surrounds discharge end of kiln. Draft is furnished by two fans. Cooler discharges to two conveyors, to open storage or to the finish-mills tanks. Gypsum is fed to same conveyor. Preliminary finish grinding is with eight ball-mills, final grinding with seven 5-ft. by 22-ft. mills. Cement is screw-conveyed and elevated to 250,000-bbl. stockhouses. Other conveyors and elevators take storage cement to packhouse to packers.—A. J. HOSKIN

Some properties of high alumina-cements from six countries. P. H. BATES. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—High alumina cements (ciment fondu) manufactured in six different countries have been examined for their properties by chemical, petrographic and physical methods. The physical characteristics were determined by strengths in the form of 1:3 standard Ottawa sand tensile briquettes and 2-in. compression cubes. Concrete was also tested in the form of 6 by 12-in. cylinders made from a 1:2:4 mix of gravel. The chemical analyses showed the cements were rather widely different from one another. Petrographic examination showed that alumina was largely present either as $CaO\ Al_2O_3$ or as the unstable form of 5 $CaO\ .\ 3\ Al_2O_3$. The wide variation in composition and the difference in constitution did not result in as great a difference in physical properties as would be expected. The effect of heat generated during setting of cement, and effect of amount of mixing water used was also studied. Increased temperatures during setting resulted in decreased strength. Increased amounts of mixing water resulted also in marked decrease in strength.—EDITORS N. I. A. T. M.

Hydraulic additions and trass-portland cement. S. I. DRUJININ. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Volcanic tuff from the Crimea and diatomaceous earth from tested sources, when ground to fineness of portland cement and added to portland cement clinker, give pozzuolana-portland cement which, when used in hydraulic structures, is more resistant to sea water and acid water than is portland cement mortar. Its mechanical strength after two months hardening in water is equal to that of ordinary portland cement mortar without hydraulic addition, using the same

quantity of sand. The best methods for rapidly determining degree of activity of hydraulic addition, are to estimate the amount of lime it absorbs, and to compare strength of mortars, with and without such addition, under conditions afforded by accelerated hardening of the test-pieces at 80 deg. C. The best method for determining optimum addition of hydraulic material is the above-mentioned accelerated or thermal method. Pozzuolana-portland cement produces a more voluminous paste than ordinary portland for same quantity of sand.—EDITORS N. I. A. T. M.

Clinker shipped 200 miles by water is ground at distributing plant. W. E. TRAUFFER. *Pit and Quarry*, Feb. 11, 1931, V. 21, No. 10, p. 57-61.—Storing and packing plant of Great Lakes Portland Cement Co., Cleveland, O., operates on clinker transported from the Buffalo, N. Y., plant of same company in self-unloading boat. At Buffalo plant clinker coolers discharge to conveyors that distribute clinker to concrete silos which feed to belt conveyor discharging clinker through a telescopic spout on a boom into boat having cargo capacity of 37,000 bbl. Five hours are required for loading, 16 hours for the voyage to Cleveland, and 8 hours for unloading. Same boat also transports cement rock from northern Michigan to the company's Buffalo plant. When unloading, boat's boom conveyor discharges the clinker into the boot of a bucket elevator from which clinker is discharged through split chute to concrete storage silos. Clinker is withdrawn through gates under silos and conveyed to mill building. Gypsum is received in hopper-bottom cars. Clinker and gypsum are ground by 30-ft. 3-compartment mill. Screw conveyors take finished product to cement-storage silos. Cement from storage is conveyed and elevated to vibrating screens, to hoppers, to automatic packers for shipments.—A. J. HOSKIN

The rate of hydration of cement clinker—Part 2 and 3. F. O. ANDEREGG AND D. S. HUBBELL. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 572-578.—Part 2—Direct comparison of hydration rates of partially hydrated commercial portland cements ground under oil checked the rates of material ground in air within 1 per cent. At 9 months the effect of calcium sulfate or calcium chloride on rate of hydration was negligible. Apparent depth of hydration of standard portland cement was 3.54 microns at 1 month, 5.12 microns at 3 months, 7.2 microns at 9 months, and was practically complete at 12 months. A high-early-strength cement and a white portland cement showed more rapid hydration than standard portland cement. Part 3—Depths of hydration were measured for particles of tricalcium aluminate, tricalcium silicate, dicalcium silicate, and calcium ferro-aluminate. Experiments were conducted at room temperatures, and corrections were made for impurities. Calcium ferro-aluminate was added in small amounts to each of the three minerals to see what effect its presence might have on the reaction rates. No definite effect was noted, probably because of great reactivity of compound itself for water. Calcium sulfate was added in part of the experiments. A few mixtures of minerals were observed, including one roughly approximating commercial portland cement. Tricalcium aluminate was hydrated most rapidly. Tricalcium silicate was not far behind, while dicalcium silicate had a comparatively slow reaction rate. Rates for portland cements were between the two silicates at ages up to 7 days. In general, mixtures reacted more rapidly than single compounds, particularly when calcium ferro-aluminate was added.—GLENN MURPHY

The relation between the strengths of cements developed by mortar specimens and concrete specimens. J. R. DWYER AND P. H. BATES. *Proc. A. S. T. M.*, 1930, Part 2, V. 30, p. 598-614.—Data are presented showing the relations between the strengths developed by several portland cements in various concretes, and in small mortar specimens. The cements were selected as displaying different, but not unusual, strength properties. The aggregates

used were standard Ottawa and Potomac R. sands, and Potomac R. gravel, limestone, and blast-furnace slag. Several water-cement ratios, and three different mixes of concretes were investigated. Ultimate strengths were obtained at 1, 3, 7, 28 days, 3 months, and 1 year. Curves showing relation between concretes and mortars at different ages are rather similar, but values of ratio are in many cases far from identical. Ratio of concrete to mortar strengths varied with age, kind of mortar specimen, and kind, grading, and proportioning of concrete. It follows that ratio will also vary for different aggregates available. These tests indicate that no such ratio can be assumed as holding for the same concretes at all ages or for all concretes at the same age. All concretes in one kind of a mortar specimen develop a strength which has a certain ratio to strength of same cement in a certain concrete, but this ratio changes with ages of testing, and with changes in kind, grading, and amount of aggregates.—GLENN MURPHY

Contribution to the problem of hardening of portland cement. FRIEDRICH F. TIPPMANN. *Zement* (Germany), Dec. 25, 1930, V. 19, No. 52, p. 1225-34.—Hydrolysis of calcium silicates of portland cement clinker, when mixed with water, is complete; end products consist of silica gels and calcium hydrate. It is unessential whether clinker is treated with little or much water. In normal cement mortar mixtures products of hydration are different due to high viscosity of system. Needle crystals formed when cement is treated with excess water are calcium hydrates which are not formed when mixtures of normal consistency are present. Calcium hydrosilicates are not formed, neither in crystalline nor in colloidal form, in hardened cement. All calcium hydrates, separated by hydration of cement, are not present in hexagonal plates or columns but show irregular lamellar shape. Unsoundness is not due to crystallization of calcium hydrates but due to amorphous calcium hydrate which is formed in later hardening periods in case of cements which contain uncombined lime. In this period, gypsum has no influence. Hardening is caused by ideal cooperation of colloid silica and crystalline calcium hydrates which are most important constituents in hardened portland cement. This calcium hydrate is the fundamental factor in hardening process. A microscopic method is described which allows observation of hardening process in cement water mixtures of normal consistency. Unsoundness due to gypsum is caused by excessive crystallization of calcium hydrate when amount of colloids present is insufficient. It is possible to make a cement, which is unsound due to lime, sound by addition of gypsum which converts amorphous calcium hydrate into crystalline form. This can be done within small limits. Several microphotographs are given.—A. E. BEITLICH

Belgian cement industry. G. BAIRE. *Revue materiaux constr. trav. publics*, Jan., 1931, No. 256, p. 17.—Belgian cement industry has shown great progress in recent years, as manifested in improvement of industrial processes and in perfection of material used. Proportioning is done with greater care and raw mix before calcination presents a more intimate and homogeneous mixture. Slurry is of greater fineness. Calcination is much more uniform and effective, so clinker obtained is generally of satisfactory appearance. Milling is also done with greater care. Numerous plants, which heretofore manufactured only natural cement, today produce portland, notably in Tournai region. Progress in specifications from 1897 to 1927—date of latest specifications—has contributed to rising standard of product. A number of plants now manufacture a high test cement whose properties are comparable to fused alumina cement. Prof. Batta, of the University of Liége, is authority for the statistical data in the article. Total production of portland cement in Belgium has grown from 250,000 tons in 1890 to 3,200,000 tons in 1929. Cement industry is one of most important industries of the country. As production greatly surpasses the national demand, the greater part of cement is exported. Belgium has been able to create and hold its markets abroad. List includes

32 Belgian plants. To guard interests of the industry, a professional Association of Belgian Portland Cement Manufacturers was formed. It exercises control over the cement products of all affiliated plants. Products are tested at University of Brussels. Of the 32 plants listed 29 are affiliated with the Association, which lends its mark only to unquestionably satisfactory products. The laboratory also is engaged in research work and attempts to popularize the acquired knowledge on cements, mortars and concretes. Abstracts of specifications are appended.—M. A. Corbin

Miscellaneous

Communication from the German material testing institutions. Mitteilungen der deutschen material prufungsanstalten (Germany). Verlag von Julius Springer, Berlin. 1930, V. 6-8.—Part A.—Development of other than Prussian material testing institutions. Includes descriptions of laboratories and equipment at the following Institutions: (a) Technischen Hochschule Darmstadt; (b) Technischen Hochschule Dresden; (c) Technischen Hochschule Karlsruhe, two departments, one for concrete and one for wood, stone and iron; (d) Technischen Hochschule München, two departments, one for building materials and one for mechanics; (e) Bayerischen Landesgewerbeanstalt; (f) Technischen Hochschule Stuttgart. Part B. Technical abstracts.—H. G. Fisk

Light mortars of pumice and Si-Stoff with portland cement. Heinrich Luftschitz. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 55-9.—Properties of cement mortars with pumice and Si-Stoff were examined. One mixture consisted of 1 part of portland cement and 6 parts of pumice (0-0.2 in.) and another one of 1 part of portland cement, 1 part of Si-Stoff and 5 parts of pumice. Si-Stoff is waste product of alum manufacture and contains large amounts of soluble silica. Analysis of Si-Stoff and pumice are given. Mortar with Si-Stoff is lighter, more dense and has better heat insulating properties than first mixture. Its coefficient of expansion is smaller and it is rust preventing. It is suitable for use with steel skeleton structures and for manufacture of light concrete pipes. Mortars for such uses should not contain more than 0.25 per cent water soluble substances.—A. E. Beitlich

Protection of construction materials against chemical and physical agencies. Schutz der Bauwerke gegen chemische und physikalische angriffe. Otto Graff and Hermann Goebel. Verlag von Wilhelm Ernst & Sohn, Berlin, 1930, 224 p.—This book is written from standpoint of engineer and chemist, and illustrations and examples are taken from experiences of the authors. It is not advanced as a comprehensive treatise, but emphasis is placed upon numerous problems encountered in field of construction and engineering. Subjects treated are: natural building stones; cementing materials, i. e., limes and cements (portland, etc.); mortar and concrete; masonry construction which includes concrete, reinforced concrete and refractory construction; metals and wood. Under each subject methods of use, or construction, physical and chemical properties, tests, etc., and known methods of protecting or prolonging the life of the material are given. Emphasis is given the proper selection of material for the service to which it is to be put, and proper engineering design and installation of the material.—H. G. Fisk

Coloring cement. *Conc. Bldg. Conc. Prod.* (England), March, 1931, V. 6, No. 3, p. 44.—Requirements for cement color pigments are color durability under exposure to sunlight and weather, fine subdivision, intensity of color when finely subdivided, and a chemical composition such that pigment does not react with cement to detriment of the color or cement. The most suitable are iron oxides (red, black, yellow, brown) manganese dioxide (black) chromium oxide (green) carbon pigments (black) ultramarine (blue). Use of organic colors is not dependable. Colors containing Prussian blue, cadmium

lithopone and zinc and lead chromates should be avoided. Brilliant or intense colors are not generally attainable. Carbon black pigments although most effective, cause serious reduction in strength and are difficult to incorporate uniformly when quantity exceeds 2 or 3 per cent of weight of cement. Black iron oxide is better pigment but the cost is rather high. Limit of 10 per cent of cement weight is usually put on quantity of pigments used, although experiments show that in certain cases this limit may be safely exceeded. Most satisfactory way of incorporating pigment is to grind with cement. For delicate tints white cement or a mixture of white and gray, should be used, and such aggregates as silica sand and marble. As far as possible the color of aggregates should harmonize with that of the cement paste. Coloring of concrete has been accomplished by penetration processes, but variations inherent in the absorptiveness of the surface usually cause some non-uniformity of color. It is usual to incorporate a fluosilicate hardener with the color producing salt, the solution being applied after the concrete has been cured and allowed to become surface dry. A final oil wax treatment is applied to fix and protect the color.—JOHN E. ADAMS

PROPERTIES OF CONCRETE

Protection of structures against chemical and physical attacks. "Schutz der bauwerke gegen chemische und physikalische angriffe." OTTO GRAF AND H. GOEBEL. 1930, Wilhelm Ernst and Son, Berlin (Germany), R. M. 22.00. Reviewed in *Zement* (Germany), March 26, 1931, V. 20, No. 13, p. 306.—Authors report in this book various kinds of destructions of structural materials and structures. Effect of chemical and physical conditions is shown by test results, tables and illustrations. Of special value are descriptions of restorations of entire structures showing useful application of experience.—A. E. BEITLICH

Effects of contraction in large concrete dams. ALBERT RENAND. *Magazine Technico* (Argentina), Sept.-Dec., 1930.—Engineers of concrete dam at Saint Marc on Thourian River having in mind dangers arising from formation of contraction cracks in concrete dams have made very careful observations on formation of cracks in that structure during construction. San Marcos dam is gravity type about 140 ft. high and contains approximately 80,000 cu. yd. of concrete. Work was authorized in August, 1928, and first cracks were observed in October, 1929. A number of other cracks were noted later and the progress of these cracks and their location in the structure is given in detail. Colored water was used to trace the penetration of the cracks —C. G. CLAIR AND M. N. CLAIR

Studies on laitance of cement mortars and concrete. SHOICHIRO NAGAI AND KOSUKE YOSHIZAWA. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), Jan., 1930, V. 33, Supplemental Binding, No. 1, p. 19-21B.—In continuation of previous work relations between compositions of laitance and water-cement ratios, admixtures or accelerators were studied. (1) Cement mortars. Results of analyses of laitance formed on 1:2 cement-sand mortars were compared with cements used. Increase in loss on ignition and lime show formation of large amounts of calcium carbonate due to carbonation of calcium hydroxide which was formed during hydration process. High SO_3 was found in cases of large w/c ratios. (2) Neat cements. Similar relations as in (1) were detected. (3) Neat cement with admixtures or accelerators. Siliceous earth, pozzuolana, lime, calcium chloride, "Cal" and gypsum were mixed in different proportions with 2 cements and their chemical composition studied. Amounts of water soluble chlorine and SO_3 were determined. —A. E. BEITLICH

Studies on laitance of cement mortars and concrete—Part 2. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), July, 1930, V. 33, Supplemental Binding, No. 7, p. 260-2B.—Results of analyses of chemical compositions of laitances formed on various concrete works using portland cement and of mortars of blast furnace slag cement and high early strength alumina cement are reported. (1) Amount of laitance on 1:2:4 concrete increased with w/c ratio. Laitance on concrete with small w/c ratio was nearly equal in composition to cement while laitance formed with large w/c ratio contained larger amounts of lime in comparison to silica. The SO_3 content increased with w/c ratio. (2) Laitance on field concrete contained large amounts of fine sand. (3) Considerable amounts of SO_3 (5 to 6.5 per cent of ignited sample) were found in laitance formed at concrete pipe making. (4) In laitances formed on high alumina cement (Ciment fondu) and blast furnace slag cement considerably larger quantities of alumina (hydrated alumina from aluminates) were found.—A. E. BEITLICH

Resistance of concrete to high temperatures. (cf. *La Journée Industrielle*, Dec. 20, 1930.) *Le Constructeur de Ciment Armé*, Feb., 1931, No. 137, p. 36.—It has been noted that reinforced concrete construction withstands fires better than steel frame structures. Highly infusible nature of cement and aggregates and no oxidation of exposed steel are advantages of concrete construction. Aside from fusibility, the compressive strength was also considered in tests made in Germany. Parallel tests of portland and slag cement mortars were carried out. Between 500° C. and 900° C., the two mortars showed similar trend of compressive strength. On the other hand, from ordinary temperature up to 300° C. slag cement mortar shows an increase in strength, while portland cement mortar shows a decrease of 10 to 20 per cent of strength values. Between 300° C. and 500° C., strength values show a decrease less rapid in the case of slag cement mortar. Exceptional heat-resisting qualities of slag cement justify use of slag as ingredient in the cement. These considerations are of interest not only in connection with fires, but in all cases of local heats encountered in the industrial processes of today.—M. A. CORBIN

Pre-determination of the composition of concrete necessary to attain a given strength and elasticity. M. ROS. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930. The formula given by Feret enables the strength and elasticity of concrete in compression to be predetermined with sufficient accuracy for practical purposes. It enables recognition of technical and economic possibilities of producing a concrete of definite and suitable quality, consistency, workability, porosity, strength and elasticity. By making use at same time of cement tests with plastic mortar according to Feret's method the whole of the wide field presented by the cement industry is governed by a unified idea. Feret's prism method enables the energy of the cement, the characteristic of the sand and gravel, and the strength properties of the concrete to be determined. In practice the prism compression strength is to be considered as correct strength for concrete. Curves for most favorable intergranular arrangement lie in the area bounded by the Fuller curve and the curve obtained by the Swiss Federal Testing Laboratory (E. M. P. A.).—EDITORS N. I. A. T. M.

Compression, tension and bending tests, cement and concrete. A. T. GOLDBECK. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930. Present practice and trends in methods of making strength tests on cement and concrete in the United States of America are discussed. It is indicated that although the 1:3 mortar briquet, made with Standard Ottawa sand and tested in tension is now standard, there is difficulty in obtaining concordant results in different laboratories. Although the compression test is still most universally used as a standard strength test for concrete, there is a

growing tendency to employ beam tests where high crossbending resistance is important as in highways. Method of making beam test has not yet been standardized.—EDITORS N. I. A. T. M.

Cement and concrete. R. M. STRADLING. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—In a report dealing principally with work carried out at the Building Research Station, Garston, England, a section on strength describes work done on effect of water-cement ratio on strength, recent increase in strengths obtained from rapid-hardening portland cements and relation between crushing strength and modulus of elasticity. Under heading of shrinkage, work on stress distribution along a reinforcing bar, influence of type of cement, percentage of reinforcement and conditions of storage are discussed. A note on the creep of concrete under load, mentions work by Faber, and briefly outlines the program of research in hand at Building Research Station. Joints in concrete and, lastly, resistance of concrete to chemical action, on which work is being done in collaboration with the Institution of Civil Engineers, also receive attention.—EDITORS N. I. A. T. M.

The crushing strength, density and water-cement ratio of concrete and mortar. E. SUENSON. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—The following investigation shows that: (1) Except that the exponent 2 can vary between 1.8 and 3.5, confirmation has been obtained for Feret's formula for plastic mortars, viz: $S = K \cdot T^2_k$, where S is the crushing strength of the mortar, T_k the density of the plastic mass, and K a constant depending on the quality of the cement, the conditions of storage and the age of the mortar. (2) In the form: $S = K \cdot T^n_k$, the equation can be generalized so as to be valid for all consistencies, from the driest to the wettest, provided the determination of T_k is postponed until solidification has occurred. (3) n has the same value for air and water storage. The reason it varies in magnitude as described in (1) remains at present unknown.—EDITORS N. I. A. T. M.

Laboratory tests and control and practice on the building site. M. SPINDEL. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Concrete of any desired quality can be rationally and with certainty produced if due heed is given to our knowledge of the testing of materials. Otherwise, concrete structures, like other structures, can show serious defects. In addition to determining strength and density, tests must be made to ascertain the expansion and shrinkage due to the rise of temperature in setting and variation of air temperature, and the resistance to frost, to attack by water, and to wear. Dam walls must be rendered impermeable by concrete facing. For the attainment of strength and density, the utmost importance is to be attached to cement, aggregates, amount of water, and method of working. The effect of these factors on the quality and cost of the concrete can be indicated by representing cement, aggregates, water and air on the Spindel four-component parallelogram.—EDITORS N. I. A. T. M.

Shrinkage of mortar and concrete. E. V. MEYER. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—A series of experiments was undertaken to elucidate the shrinkage of light concrete, especially the so-called cellular concrete. As is well known this material is produced by mixing together a cement paste and a foam. To attain sufficient strength in this material (which is produced with a density varying from 16 to 70 lb. per cu. ft.) a relatively rich cement mortar must be used, which results in serious shrinkage on drying out. The importance of the various factors which affect this shrinkage is investigated, and the results are as follows: (1) Type of Cement. As was to be expected, there are important differences between various cements. Finer grinding of the same cement gives rise to more pronounced shrinkage. (2) Mixing proportions. By increasing the amount of added sand, the shrink-

age step by step diminishes. This decrease is quite regular for cellular concrete, and the author has advanced a formula for the shrinkage of cellular concrete of different proportions. (3) Added Water. It is established that the shrinkage is increased by increasing the amount of water used. (4) Foam Liquid. The foam liquid has no effect on shrinkage. (5) Storage. Keeping the concrete wet over a long period, followed by intensive drying, increases the ultimate shrinkage; shrinkage is diminished, on the other hand, by mild drying out. The shrinking of cellular concrete products is investigated. Finally it is shown that treatment of the material with saturated steam at 100-180 deg. C. results in a considerable reduction of the shrinkage.—EDITORS N. I. A. T. M.

Influence of frequently repeated load on the formation of cracks and the cracking—Safety of reinforced concrete beams—Einflusz haufig wiederholter Belastung auf die Riszbildung und Riszsicherheit von Eisenbetonbalken. HEIM. *Dissertation an der Technischen Hochschule Karlsruhe* (Germany), 1930, 71 p.—In the first section the author reviews previous investigations. Second part takes up special subjects such as effect of variation in speed of application of load, apparatus, test specimens, and preliminary tests. Third section deals with static loading of reinforced concrete beams and includes measurements of elasticity and determination of tension on columns and beams. Formation of cracks in reinforced concrete beams is also taken up. The fourth section treats the effect of frequently repeated load and includes the investigation of concrete beams with and without reinforcing. Conclusions include: (1) Young concrete under frequently recurring load attains a permanent deformation which exceeds the elasticity. (2) A repeated load which does not exceed half of the static load causing cracking, will not cause cracking even after one million load cycles. With a load equal to three fourths of that causing cracking under static loading, cracking will occur under repeated load. (3) If cracks have already appeared under a certain static load, on changing the same load to a constantly repeated load, a condition of equilibrium will be attained in the cracks before the millionth load cycle is reached.—H. G. FISK

Comparative study of the strength of mortar and concrete under tensile, bending, shear, indentation and compressive stresses. R. FERET. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—The author refers to and confirms the fact that it is first necessary to distinguish between two classes of fundamental resistances, firstly, resistances to compression, indentation and shear, which are proportional to each other, and secondly, resistances to tension and bending, which are also proportional to each other, but bear no fixed relation with the first class. Author then shows, both from personal experience and from analyzing minutely the results of various other experimenters, that in general, for series of mortars and concretes made with the same cement and differing in only one factor, resistance to tension (or to bending) increases less rapidly than the resistance to compression, and varies approximately with 2/3 power of the latter. Otherwise for mixtures of various composition, ratio between first and second of these resistances is approximately proportional to that between the quantities of water and cement used. Disagreement between two classes of resistance is due principally to fact that surface layers of test pieces, which are of first importance in rupture under tension and bending, seldom have same composition as the core. This is due as much to unequal distribution of materials against the walls of the mold and in interior, as to a hardening which varies for various regions, according as the surrounding conditions have a more or less direct influence. Apart from this, resistances are variously influenced by various internal stresses which vary according to distance from free surface, and which result from changes in volume due to chemical, thermal and hygroscopic action. It seems, however, that the tensile and bending tests do not give very accurate information regarding similar forces to which concrete is sub-

jected in practice, although the figures given by compression tests are fairly proportional to the actual strengths.—EDITORS N. I. A. T. M.

Cracks in concrete. M. E. ROSSBACH. *Revue materiaux Constr. Trav. Publics*, Jan., 1931, No. 256, p. 12-17.—Development of cracks in plain and reinforced concrete is consequence of shrinkage of cement and of the low resistance of concrete to elastic stresses, other than compression. As, due to foregoing, concrete is depended upon only in its resistance to compression, its other properties are frequently neglected and a concrete is classed entirely in accordance with its compressive strength. Our daily experience shows failures of concrete in compression are rare, in spite of unreasonable loadings and irrational proportioning of materials. Using standard portland cement and an aggregate of river origin or of porphyry, gives concrete well able to resist usual stresses with required safety. Yet cracking of concrete occurs all too frequently and is main grievance against this material. While compressive strength remains the primary requisite of concrete, attention should be paid to its other qualities. A ductile concrete, least affected by cracking and friction, should be sought. The deplorable appearance of cracks on structures, the menace to life of structure and to reputation of reinforced concrete in general should be kept in mind.—M. A. CORBIN

The cause of poor agreement between tensile strength of neat cement and sand mortar strength. KATSUZO KOYANAGI. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), Nov., 1930, V. 33, Supplemental Binding, No. 11, p. 425-7B.—Experiments by R. Meade, G. Haegermann and J. Iguchi are discussed. Based on author's investigations, strength of neat cement is just as much affected by amount of mixing water as is mortar strength. Best results in each case are obtained with a certain fixed quantity. Cements which pass water test but not boiling test have in general very high tensile strength in neat specimens and low sand mortar tensile strength. Fineness affects strength of neat cements: coarse samples show in general greater tensile strength than fine cements (while mortar strength is mostly improved by fine grinding). Factors which increase strength of neat cement have adverse effect on mortar strength.—A. E. BEITLICH

Shrinkage of concrete in actual structures. WILLIAM M. BASSETT. *J. Boston Soc. C. E.*, March, 1931, p. 67-81.—Change in volume of concrete must be expected and New England Power Construction Co., has tried to design the Fifteen Mile Falls Lower Development so as to reduce the damage from such volume changes to a minimum. Spillway structure was poured in sections 45 ft. long after a study taking into consideration the possible shrinkage of the concrete as 0.055 per cent of its length, curvature of the bulkhead plant, restraint of bed rock and capacity of concreting plant. A field survey made approximately one year after pouring of concrete showed six out of eight of the 45-ft. sections had intermediate cracks. The cracks appeared in less bulky sections and indicated that if sections something less than 30 ft. long had been used, intermediate cracks would not have formed. Spillway channel slab used for protection of earth bank of spillway channel was built on concrete ribs spaced about 20 ft. centers. Elimination of irregularities on the surface was important as water velocity is high and slabs were therefore designed with 0.5 per cent steel to prevent cracks forming. Recent field examination shows a crack between slabs and ribs and also between slabs and toe beam. Theoretical width of cracks between slab and rib is 0.06 in. and actual width was found to be about same. Theory in design and procedure in construction of intake structure are described in detail. Every effort was made to anticipate and overcome effects of volume change that might result in a crack through which seepage would occur.—MILES N. CLAIR

Tests of plain and reinforced Haydite concrete. (See Materials—Aggregates.)

The strength of Swiss portland cement. (See Materials—Cement.)

Engineering Design

Bridges

Longest multiple-arch concrete highway bridge to span Susquehanna river. Charles M. Nelson. *Eng. Con.*, March, 1931, p. 119-122.—River crossing between Columbia and Wrightsville, Pa., has a total length of 7,374 ft., including approaches. The spans of bridge are on a tangent with exception of first span on each side. The river structure consists of 28 concrete arches with three ribs each spaced 17 ft. 9 in. on centers, parabolic intrados and extrados, rise of 32 ft. and 27 ft. 4 in., respectively, and a clear span of 185 ft. The rib section is 7 ft. wide and varies from 3 ft. 10 in. at the crown to 7 ft. at the springing. Each span is divided into 11 panels by transverse spandrel walls. Arch ribs are joined at alternate panel points by concrete struts. Floor system is carried on concrete spandrel columns 2 ft. 6 in. by 4 ft. along the line of outside ribs and 2 ft. 6 in. by 3 ft. 6 in. along the line of the inner ribs. Floor panels are formed by longitudinal and transverse floor beams and are 16 ft. 10 in. by 17 ft. 8 in. Floor slabs are of two-way reinforced concrete. An unusual feature in the floor design is the arrangement and design of expansion joints. Super-structure will be completely separated by open joint 1½ in. wide in floor system and 1 in. in railings at center of first and fourth panels from each abutment or pier. Thus each arch floor system will be cut at four points between piers. Joints are to be filled with bituminous filler and flashed with copper. Roadway is 38 ft. between curbs. There is one 6-ft. cantilevered sidewalk. The approach structures consist of 18 50-ft. concrete girders and two 80 and 85-ft. steel girders, all on concrete piers.—N. H. Roy

Buildings

Design of a 15-story apartment building in Vienna (Austria). *Zeitschrift Oester. Ing. Arch. Vereins* (Austria), Jan. 9, 1931, V. 83, No. 1-2, p. 1-2.—New reinforced concrete skeleton structure of unique design rests on one single reinforced concrete slab foundation. First and second floor are occupied by stores and offices. Apartments are located from third to tenth floor and restaurants occupy tower structure.—A. E. Beitlich

Storage building with precast roof arches. Mangold. *Beton u. Eisen* (Germany), Jan. 20, 1931, V. 30, Heft 2, p. 37-38.—The city of Munich received 37 competitive designs in reinforced concrete, steel and timber for a storage shed for iron water pipes. Reinforced concrete design of Wayss & Freytag A. G. was selected as most satisfactory and most economical. Building is a one-story structure 345 ft. long by 76 ft. wide and 56 ft. high. A crane runway extends the length of structure and 69 ft. beyond, spanning the adjacent street. The girders are roughly I-shaped and carry the crane rail on the lower flange. Girders are about 12 ft. deep with a web thickness of 1 ft. 4 in. and a chord thickness of about 4 ft. They are carried by supports spaced at 69 ft. and with these supports form a series of three three-legged bents along each longitudinal wall. These bents are connected by hinges in the crane girders. Arched roof members, about 1 ft. 6 in. deep with a rise of 15 ft. 9 in. and spaced at 8 ft. 7 in., were precast on the ground and lifted into place by crane. Crown reinforcement was designed to act somewhat as a hinge, and this section and two ends of arch were not concreted until arch was in place. Alternate arches were provided with tie rods to take up horizontal thrust. A reinforced member

extending the length of structure connected arches at crown. The light weight concrete slab between the arches also exerts a stiffening effect. Only scaffolding necessary for roof was a line of timber framing along center of building, supporting crown. Skylights are built between the arches and stop about 10 ft. short of eaves. Glass area is 25 per cent of total roof area. To enable loaded crane to pass out of building a sliding panel between the crane girders and a large folding door are provided at one end.—A. A. BRIELMAIER

DAMS

Cellular corewalls for embankment dams. H. C. HOWELL. *Eng. News Record*, Feb. 26, 1931, V. 106, No. 9, p. 359.—A new type of cellular concrete corewall for embankment dams has been developed by Mexico office of the J. G. White Engineering Corp., consisting of a row of vertical arched cells each of which drains into a longitudinal conduit at the base. Conduit discharges into outfall drains at convenient places. Important features are (1) downstream portion of dam is kept dry; (2) cells and drains permit inspection; (3) cells form stiffer structure than a solid wall (4) arched sides are more economical than flat ones because tensile stresses and necessity for reinforcing steel are eliminated. The idea for the cellular corewall was put into practice in 1917 to 1919 in the construction of the Requena Dam about 50 miles from Mexico City.—D. E. LARSON

Design and economy of concrete and reinforced concrete dams. E. PROBST AND F. TOELKE. *Zeitschr. d. Vereins deutsch. Ing.* (Germany), 1930, V. 74, No. 13-15, 15 p.—Paper, which was presented at second world power conference in Berlin (Germany), 1930, pictures present state of dam construction considering especially safety and economy. Ways are shown to prevent failures. In general, curved form of gravity dams does not increase stability of structure unless contraction joints are filled and treated in manner similar to that in construction of Pardee dam (cf. *Eng. News Rec.*, Feb. 14 and March 21, 1929, V. 102, No. 7 and 12, p. 258-62, 457-8). Study of cross-sections of numerous dams shows predomination of triangular type. Relation between width of dam at bottom and total height is characterized. Development of arch dams is illustrated. Corfino and Turrite dams in Italy are of elastic arch type. Latter dam has constant outer radius of curvature. Wall at crest is 13.8 ft. thick at center and this dimension increases to 19.7 ft. at both sides. As examples of constant angle arch dam type are described Big Santa Anita, Pacoima and Diablo dams (cf. JOURNAL, A. C. I., Sept., 1930, *Proceedings*, p. 1-64). Latter has two gravity abutments. Attention is called to gradual thickening of foundations on downstream side of dams. Mesnager builds a number of arch dams, decreasing in height, between each two of which a water reservoir is formed. Walls can be built relatively thin. A new type of dams designed by authors is described. Its lower part is of gravity type while upper part is arch structure. Gutzwiler, Figaro and Ziegler systems are sketched. Concrete structure of Stony Gorge dam in California is of Ambursen type. (cf. *Eng. News Rec.*, July 11, 1929, V. 103, No. 2, p. 46-51.) Main advantage of this type is even distribution of loads on underground. Buttress spacing over 18 ft. is not economical. Big Dalton dam is described as an example for multiple arch type dam with double wall buttresses (cf. *Eng. News Rec.*, Dec. 26, 1929, V. 103, No. 26, p. 994-7). Formation and causes of cracks are discussed in connection with those structures. Third part of article deals with safety and economy and gives general directions concerning foundations, calculations, structural materials and recommendations for research work in dam construction. Great emphasis is placed on temperature measurements in interior of large concrete masses.—A. E. BEITLICH

Gravity dams arched downstream. B. J. LAMBERT. *Proc. Am. Soc. C. E.*, March, 1931, V. 57, No. 3, p. 425.—It has been a common practice to arch masonry gravity dams upstream with the thought that additional factor of safety was introduced by arch action. When walls of canyon are not sufficiently rigid to support thrust, there is no gain from arch plan over a straight gravity dam. In such cases stability of a straight gravity dam against overturning may exceed that of a gravity dam of same section that has been arched against water load and a dam arched downstream may have a stability that exceeds either. In illustrating this point the author makes use of measurements and calculations from a 1/240 scale model of the late St. Francis Dam. In his analysis he treats the structure as a unit and shows that the portion of dam about which overturning would occur (the pivot) lies farther from the centroid for gravity section than for section arched upstream and farther for section arched downstream than for either. Length of dam that can act as a pivot again favors reverse arch. The objection that reverse arch plan would tend to open up rather than close joints is considered and it is suggested that anchored eyebar chains under initial stress could be embedded in concrete of dam to develop any desired initial compression in the concrete. While admitting unorthodox nature of recommendations the writer urged that in any case preliminary studies should be made on a model of proposed site and structure.—H. J. GILKEY

Problems in concrete dam design. D. C. HENNY. *Eng. News Record*, March 12, 1931, V. 106, No. 11, p. 431-435.—Wide differences of opinion regarding fundamentals of safe and economical design of concrete dams still exist. Danger inherent in uplift is now thoroughly realized by dam designers and builders and means of reducing its effect are: roughening rock base and concrete surfaces to increase adhesion and resistance to water creep and sliding; grouting foundation; introducing drains in foundation and mass concrete; inclining or stepping construction joints upwards in downstream direction; using water stops in these joints near upstream face. Grouting upstream toe of dam does not guarantee against uplift but helps to diminish it. Drains in foundation rock and in concrete are now commonly provided, but requirement that these be straight and accessible is too often neglected. Principal cause for extensive cracking in large dams may be tendency of cement manufacturers to increase fineness of grinding. This affords advantages in workability, early strength and form cost, but in mass concrete important effect is greater heating and consequently an increased liability to cracking. Heat control may be effected by amending cement specifications. Another method of heat control consists of artificial cooling of interior by water circulating through pipes. Problem of heat stresses and cracking in dams can be solved by use of precast concrete blocks. This would reduce cost of form work, but cost of handling would be high.—D. E. LARSON

MISCELLANEOUS

Stands of novel design at English race course. *Concrete*, Feb., 1931, p. 95.—The stands at Northolt race course are of steel and concrete. The roof deck is carried by cantilevers of steel encased in concrete. Cantilevers extend 60 ft. forward from the back of the stand.—N. H. ROY

Short-cuts in structural design. JAMES R. GRIFFITH. *Concrete*, Feb., 1931, p. 45-48.—Unbalanced reinforcing is discussed for the case of concrete beams and slabs whose dimensions have been determined by considerations other than that of producing maximum allowable stresses in both concrete and steel. A chart is given for determining actual concrete stress and area of steel required, for a breadth of 12 in. and f_s of 18,000 lb. per sq. in. A second chart is for f_s of 20,000. Three values of n are given.—N. H. ROY

Calculation tables for hollow concrete brick ceilings. HANS KORF. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 61-3.—A graph is described which enables one to read directly thickness of bricks, total thickness of ceiling and size of reinforcing when span, load and type of support are given. Several examples explain use of graph. It can be applied to ceiling systems by Ackermann, Kleine, Wenko and Remy. Reverse reading of table gives greatest span for certain ceilings of given dimensions.—A. E. BEITLICH

Structural design and construction of concrete improved in 1930. *Concrete*, Feb., 1931, p. 16-18.—A very interesting bibliography of technical papers, results of tests, committee reports, and contributions to the field of design for the year 1930.—N. H. ROY

Concrete bulkheads for creek channel improve Jacksonville swamp area. *Eng. News Record*, Jan. 8, 1931, V. 106, No. 2, p. 70-72.—The unsightly Hogans Creek Swamp at Jacksonville, Fla., has been changed to an attractive waterway built of concrete as result of a $500,000 improvement program recently completed. Design of channel structure finally adopted provides for a box 20 ft. wide on two rows of wood piling, 18 ft. apart, with piles driven on 5-ft. centers. A 6 x 8-in. cap runs longitudinally over top of each row of piles. Precast concrete T-beam struts span the two rows of piling on 5-ft. centers. Each end of the struts carries a 6 x 8-in. longitudinal wooden stringer, and upon these stringers rest precast slab units that form sides of box. Two-in. planking is then placed longitudinally under the struts and the bottom of the box is filled with sand to the depth of 2 in. above the top of the strut, which provides a porous floor system for admission of groundwater to channel. Bulkhead panel lengths vary from 6½ to 9 ft. and are precast in 2-, 4-, and 6-ft. widths. The 6-ft. widths are used generally except on curves. Gang molds are used in casting yard where bulkhead panels and T-beams are cast. This yard permits casting of 150 lin. ft. of bulkhead and 21 T-beams at same time. A central mixing plant provides all concrete for the job, for both precast and cast-in-place work. A minimum strength of 3,000 lb. is specified and obtained under laboratory conditions.—D. E. LARSON

Reinforced concrete. S. M. DIXON. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Exact knowledge of the properties of concrete especially in reinforced concrete structures is meager. Innumerable tests on compressive strength of concrete have little advanced accuracy in design of reinforced concrete beams. Experimental work on shrinkage, expansion, modulus of elasticity and plastic yield is still needed. Disintegration of plain concrete under repeated loading makes continued security of some reinforced concrete structures problematical. Engineers are much interested in stresses induced in steel during setting of surrounding concrete, and tests so far are hardly conclusive. Recent design has been much influenced by special light aggregates or by using aerated cement.—EDITORS N. I. A. T. M.

Tests on reinforced concrete structures. R. SALIGER. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—In the years 1927-1929 the following tests were suggested by the writer of this report and performed at the Engineering Testing Laboratory of the Polytechnical Institute of Vienna. In 1927 tubes for the power-station of Blumau and also a new shellroof system, suggested for the barrelwashing-shop of the brewery of Schwechat, were tested. Both these series of tests were performed on full-sized models. In 1928, experiments were carried out on concrete beams reinforced with Isteg-steel. They showed the influence of two round iron bars twisted cold, while during twisting the ends of the bars are rigidly held in their places. These experiments were followed by tests on beams of concrete, reinforced with high-quality steel, and others on pillars of concrete reinforced with cast iron and corded with steel. In 1929 a series of tests on concrete pillars reinforced with high-quality steel, yielded important results.—EDITORS N. I. A. T. M.

Combined compression and bending in reinforced concrete. E. M. ROYDS-JONES. *Concrete Constr. Eng.* (England), Feb., 1931, V. 26, No. 2, p. 144-149.—Charts are given for determining thickness of concrete and amount of reinforcing steel necessary in slab or beam subjected to combined bending and compression for cases where concrete is not stressed to a specified allowable stress. This is the case where the size of member is fixed already by other considerations. Example shows application of charts. Total moment of external forces about steel is obtained for beam of known dimensions, bending moment and end compression. Then $H = M/bd^2$, where M = moment of forces about steel, b = width and d = depth of beam. From graph value of stress in concrete obtained for calculated value of H and selected value of stress in steel. Curves are given for different stresses in steel. With value of stress in concrete thus obtained a second graph gives values of k (ratio of depth to neutral axis, to depth to steel) for given allowable stress in steel. Then compressive stress in concrete $= 1/2 f_c kbd$, tension in steel and required area of steel are obtained. Examples illustrate the usual general case, case where concrete dimensions are too small, and where no tensile steel is required.—JOSEPH MARIN

Deformation of the reinforcement as a measure of its value in ferro-concrete. E. H. F. EMPERGER. *First Communications of N. I. A. T. M.* Group B, Zurich, 1930.—Investigation of ferro-concrete has been limited to measurements under permissible load and determination of breaking loads. The connection between the two has been represented by a theory which cannot be regarded as free from objection: that enclosed steel is constrained to deform with the concrete, and that the measurement of this deformation in the case of the reinforcement of a beam in tension enables the stress taken by this reinforcement to be determined, so that the variation of the quantity n as load increases can be evaluated. The relationships under compression are much simpler. Parallel experiments on concrete columns, with and without reinforcement, are used to determine the increase in strength due to the reinforcement under same deformation of concrete. These tests show that reinforcement which is not proof against buckling does not withstand compression as well as the surrounding concrete. The feeble round steel rods yield sideways, and do not contribute to the strength of the member to an extent corresponding to the magnitude of their deformation, even when they are held within a circular reinforcement. The most important result of this research is that by the use of a buckling-proof reinforcement of cylindrical profile, within a circular reinforcement, full utilization can be made of the strength of the steel under compression up to the yield point, with similar utilization of the concrete. It is also possible in this way to utilize the much higher yield points of high-strength steels.—EDITORS N. I. A. T. M.

Solution of statically indeterminate structures by Begg's method. K. SCHOECHTERLE. *Beton u. Eisen* (Germany), Nov. 20, 1930, V. 29, Heft 22, p. 406-410.—In brief review of theory and application of Prof. Begg's mechanical method of solving statically indeterminate structures, the last section describes experience of Stuttgart office of Bridge Dept., German Government Rs., with this method. This office, in 1927, designed and erected a concrete arch bridge at Weikersheim, with a span of 90.0 ft. and a rise of 27.6 ft. It was decided to compare results of a mechanical solution with completed analytical analysis of this structure. Preliminary tests with models bearing a size ratio of 1:25, 1:50 and 1:100 to the original structure showed that the 1:50 ratio was most satisfactory for this arch. A model in 0.08 in. was accordingly made in this size ratio. Influence lines comparing results of analytical and mechanical solutions for moments, shears and thrusts at crown and springing show remarkably close agreement. In this case mechanical solution must be considered equal to analytical. Thorough and critical investigation made of this method led to conclusion that it is applicable to practical use.—A. A. BRIELMAIER

Nomogram for calculating the compressive and tensile stress produced by a force acting on a rectangular section. JORGE QUIJANO. *Ingenieria* (Mexico), Jan., 1931, V. 5, No. 1, p. 8-9.—An alignment chart, one of a series of value in design of concrete, for determination of stress at any point on a section subjected to a force perpendicular to that section, acting on an axis of symmetry. Left hand scale or line $\frac{12\ pr}{a^2}$ intermediate scale R; and right hand scale P/ab all in metric units. a and b are dimensions of the section on which the force P acts. p and r are abscissa respectively of point where force P intersects section and of point where stress R is desired.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating the unit shear by means of bending moments. JORGE QUIJANO. *Ingenieria* (Mexico), Jan., 1931, V. 5, No. 1, p. 16-17.—An alignment chart, one of a series of value in design of concrete, for the determination of shearing stress at any section of reinforced concrete beam. Left hand line or scale bd; intermediate scale $\frac{M_A - M_B}{S}$ and right hand scale u. b and d are the dimensions of the section, M_A and M_B the moment at points distant S apart and u is unit shearing stress, all in metric units.—C. G. CLAIR AND M. N. CLAIR

Short cuts in designing reinforced concrete beams. ODD ALBERT. *Civil Engineering*, April, 1931, V. 1, No. 7, p. 635-37.—The amount of work involved in the design of concrete structures may be reduced either by using more simplified formulas or by use of tables and charts. Formulas are presented for rectangular beams in a simplified form making it possible to solve for depth with one movement of slide rule runner. Charts for the design of beams and flat slabs of reinforced concrete are also presented. In working out these formulas and charts, it was assumed that variation of stress to deformation of concrete in compression is a straight line and concrete in tension was neglected.—D. E. LARSON

Circular culverts. W. S. GRAY. *Concrete Constr. Eng.* (England), March, 1931, V. 26, No. 3, p. 213-219.—Uncertainty of distribution of loading on an empty culvert or subway is pointed out. Upward reaction differs according to method of construction adopted. There may be a uniformly distributed upward reaction, line contact and concentrated reaction, or a reaction distributed over a saddle in contact with a segment of pipe. In small culverts the reinforcement will generally be the same at all parts. Expressions given for combining moments due to uniformly distributed downward, upward, and lateral loads, moments due to active earth pressure can be found. Expressions are given for case of loads concentrated at opposite ends of vertical diameter. Moment thrust and shear for any section are obtained, similiar to calculations for stresses in a circular ring. Castigliano's principle of least work is applied neglecting the internal work due to normal and shearing stresses. Expressions are given for bending moment and thrust for any point due to the dead load of pipe and for uniformly distributed load. Curve shows variation in moment coefficients along circumference of pipe. Curve also plotted showing variation of thrust coefficients along circumference of pipe. Expressions for moments and thrusts are given for cases of two radial forces symmetrically arranged and of two vertical loads symmetrical about vertical axis of pipe.—J. MARIN

ROADS AND PAVEMENTS

Concrete street construction in Camlenwell, Victoria. C. M. P. NEYLON. *Commonwealth Engineer*, Dec., 1930, V. 18, No. 5, p. 188-193.—Following a visit of S. M. Goldsworthy to America in 1924, the council has undertaken an extensive program of concrete street construction. Concrete

roadway is 20 ft. wide between curbs which are 6 in. wide and 6½ in. high (to permit of 2-in. surfacing of asphalitic concrete at a later date). Crown is 3 in. and thickness of roadway 7 in. Foundations are prepared by mixing ashes and sandy loam 3 in. thick and adequate drainage of the subgrade is insured by drains. Importance of reinforcements at weaker points as corners, joints, etc., is stressed. Thermal expansion and that due to variations in water content were allowed for at the rate of ½ in. per 100 ft. Joints were made slightly wider in the curbs. The mix was 1:5.6 by volume, based on Abrams' fineness modulus but was altered to suit variations in material. Slump was about 1 in. and gave 28-day strengths of 3,300 to 3,800 lb. per sq. in. Screeding was done against the grade. Changes of grade were eased off by ordinary vertical parabolic curves.—C. A. JORSS

Concrete roads, past, present and future. S. H. WALKER. *Structural Eng.* (England), Feb., 1931, V. 9, No. 2, p. 55-64.—In a discussion of progress in the art of road building the disadvantages of joints is pointed out. The Illinois Bates Section is shown with excerpts from various reports presented at Sixth International Road Congress. Since British roads are subjected to extremely heavy traffic, with axle loads up to 28 tons and with steel or hard rubber tires, the importance of smooth joints is far greater in Great Britain than in America. A patented interlocked joint was developed to improve joint efficiency. An apparently crackless concrete road, free from opening and closing joints, has been built from a design of C. P. Courtenay, Borough Engineer, Greenwich. Details are given for this road which consists of a reinforced grooved concrete base topped by a 9-in. thick plain concrete road crust. Grooves which key road crust to base cause invisible closely spaced cracks. Preparation of the sub-grade varies with the soil conditions. Mr. Courtenay's design has been patented as "Anchorete" concrete road. Further application of "Anchorete" plan has been made to resurfacing of old roads.—V. P. JENSEN

The bureau of public roads. THOMAS H. MACDONALD. *Civil Engineering*, March, 1931, V. 1, No. 6, p. 494-496.—Article contains an interesting description of studies of cement concrete made by the Bureau of Public Roads, including measurements of expansion and contraction of concrete due to temperature and moisture change; determinations of effect of alternate freezing and thawing, both during and subsequent to setting period; determinations of effect of time of mixing in full-size paving mixers on strength and uniformity of product; the effect of alkali salts in soil and water on concrete; and means of protecting concrete against alkali attack by bituminous paint coats. Other useful studies deal with effect of kind and proportion of coarse aggregate upon strength and density of resulting concrete. Nearly all concrete pavement curing methods thus far proposed have been tested in an investigation carried out on a section of Tennessee highway 15 miles long. One-half of pavement throughout entire length of road was cured with wet earth and character and behavior of concrete thus treated have been taken as standard of comparison by which relative effects of other curing methods used on the other half of pavement have been determined. In an effort to rationalize design of concrete road slabs, the bureau has experimentally evaluated practically every factor involved except important one of subgrade support. Experiments have yielded all necessary data with reference to contraction, expansion, and warping of slab under various conditions of temperature and moisture. Coefficients of friction between slab and various subgrades have been determined, and distribution of pressure to subgrade from loads applied at various points on pavement measured by soil pressure cells. Related effects upon pavement stresses of joint spacing, subgrade friction, and kind and position of steel reinforcing have been fully explored; and, finally, distribution of stress induced in slab by applications of static loads and impact forces has been studies to a definite conclusion, qualified only by undetermined influence of subgrade variation.—D. E. LARSON

ARCHITECTURAL DESIGN

Build business district of concrete in residential suburb. *Concrete*, March, 1931, V. 38, No. 3, p. 16-17.—Monolithic concrete, concrete masonry and stucoo with cast stone trim are being used in construction of Texas suburban commercial center, in conformity with architecture of Spanish village. Arched entrances and stone doors are reproduced in cast stone with travertine finish. All stone is dry tamped. Finish is obtained by sprinkling rock salt into molds before facing is placed and tamped. During curing salt is dissolved by frequent washing. Holes remaining provide rugged, porous finish. Pleasing color obtained by using white portland cement with coarse, whitish Texas stone and river sand, proportioned 1:1½:1½.—C. BACHMANN

Form details and practice developed for architectural concrete. *Concrete*, March, 1931, V. 38, No. 3, p. 39-42.—Clear cut lumber and fiber board form lining were used in Pacific Coast buildings. Lining was used on all form work intended for flat wall faces. Corners were rubbed down with sandpaper and surface coated or greased with form oil and wiped off. Vertical beveled strips used to obtain fluting were made from kiln-dried spruce, planed, and grain filled with lead and oil before being nailed to forms. Mixing proportions ranged from 1:2½:3½ to 1:2:3, with slump of 4 to 5 in., and 7 to 8 in. for architectural concrete. Horizontal straight lines were obtained at day's work plane by tacking dressed 2 by 2-in. horizontal strip within wall form, against outer face, and at level where concreting is to be stopped for day. Fresh concrete was then placed to a level about ½ in. above lower face of strip and sloped downward toward inner face of wall. About two hours later strip was removed and entire surface of concrete cleaned with stiff broom. Forms were removed six or seven days after concreting. Following removal, exposed surface were given wire brushing to remove fins and films. Waste molds were left in place until building was practically completed when decoration was cleaned with wire brushes and allowed to dry out until uniform in color, followed by fine dash of colored stucco or oil stain, for polychrome effects. Patching mortar proportions were same as for concrete.—C. BACHMANN

Technique and form in cast stone. E. HEMMINGER. *Beton u. Eisen* (Germany), Oct. 20, 1930, V. 29, Heft 20, p. 362-364.—Artificial stone with its manufacture based on wide experience and scientific investigation must be accepted as a satisfactory building material—technically and economically. A recent pamphlet, (*Der Betonwerkstein* by Bund der Deutschen Betonwerke E. V.), is referred to for detailed information on the properties and advantages of artificial stone. Adaptability of cast stone has made it especially suited to lines of modern architectural design. Effects obtained with various aggregates offer a wide range of possibilities in color schemes. Different surface treatments present many variations in appearance. Cast sculpture or cast ornaments may be employed to emphasize portions of a structure or to relieve an otherwise monotonous elevation. A comparatively large market is presented in the field of garden and cemetery architecture. Benches, springs, statues, arbors and monuments may be constructed of cast stone since the modern product has shown itself to be a durable weatherproof material. The time of cramped imitation of natural stone is past and the new material is developing a style and form of its own. Its nature makes it available for designs for which other building elements are unsuited. The latest use of cast stone which has been forced to overcome some resistance on the part of conservative authorities is in church architecture. The Frauenfrieden Church in Frankfurt, A. M., is one of the striking instances of the employment of cast tiles for exterior facing. Interior equipment such as altars, baptismal fonts, etc., may be designed very effectively in cast stone.—A. A. BRIELMAIER

Field Construction

Bridges

Ancient bridges and their preservation. Steven Norris. *Roads Rd. Constr.* (England), March 2, 1931, V. 19, No. 99, p. 79.—Various means of renovating and reconstructing ancient bridge into a condition of traffic efficiency were applied without detracting from the bridge's authenticity as an ancient monument. First method is that of lining voussoirs of arch and faces of the abutments with a strengthening shell either of reinforced concrete, concrete or masonry. New arch is formed below existing arch and the existing abutment thus stiffened, the new facing taking some of thrust. New footings for lining wall will probably be required but if not possible then corbels may be tied into the old abutment to take reactions. Finished lining may be stepped or splayed off 4 or 5 in. from the external face of bridge. The second method consists of removing road surface down to arches, repairing and rejointing them where required and packing concrete from haunches to crown level, and reinstating road surface. If this method raises crest of bridge inconveniently high, coping of parapets may be raised by inseting a course or two of well weathered masonry from old local walls, etc. Further stiffening of the arches below may be supplied by erecting two or three arch ribs in masonry, concrete or reinforced concrete, below the arches; the ribs being well keyed to the existing masonry. The third method consists of the demolition of one half of the bridge floor or brick arches at a time, without in any way disturbing the external faces or parapets. The arches are then rebuilt entirely in any form desired. The abutments and piers should be thoroughly overhauled, repaired and grouted where required before erecting new arch centering. Arched slabs between piers and abutments, or springing from new lintels laid on the pier walls, or reinforced concrete girders at 2 or 3 ft. centres, supporting flat precast slabs may be employed. The fourth method is for use when the bridge is too frail to act in any way towards supporting modern traffic. The old bridge is retained merely as a shell to the new and entirely self-supporting structure built above the old arches. A new reinforced concrete single span arch is built and supported on new abutments behind old abutments. Old bridge may be employed to support weight of the centering. In fifth method when the old bridge only needs considerable stiffening, two or three narrow trenches are cut through the floor of the bridge. Centers are erected and the new separate single span arch ribs are constructed springing from the old abutments or new ones behind. —John E. Adams

Buildings

Experiences with aerocret-gas-concrete. Werner. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 63-4.—Experiences with a light concrete during construction of large dwelling house, are given. New material shows great advantage over ordinary concrete for certain purposes. Insulating properties for heat and sound are exceptionally good. Handling of large size stones is very easy. Plaster adheres very well to porous material.—A. E. Beitlich

Well foundations for construction of several buildings of German Museum in Munich (Germany). Karl Baessler. *Report 33rd main meeting German Concrete Association,* March 17, 1930, p. 409-30.—Design and method of construction of foundations for 2 large reinforced concrete buildings embodied new interesting features. Location of structures is on island in river Isar where underground conditions are extremely bad. Several layers of marl, sand and gravel, with 12-ft. loose materials, are on top of sound blue clay. Drillings were made to depths of 82 ft. to investigate bearing capacity of underground. Great variations in this direction led to construction of concrete

piers in open wells. Cylindrical piers are from 3.28 to 4.10 ft. in diameter and 32.8 to 39.7 ft. deep, belled out at base to 9.8 ft. in diameter. Twelve rotary tower drills traveling on top of wells were in operation. Steel tubes were from 4.9 to 6.6 ft. long and thickness of walls was 0.4 in. Loose materials were removed by streams of water, while grabbucket dredge was used for harder strata. Bell shaped excavations for footings were made by hand with compressed air shovel. Concreting was by continuous shoveling of plastic mixture until ground water level was reached. After pulling steel tubes remainder of concrete was placed thus preventing washing out of cement by ground water. Tops of 500 piers were connected with horizontal reinforced concrete girders which support basement walls. Only 2 piers had to be replaced.—A. E. BEITLICH

Winter concreting with use of gas burners. V. SMITKA. *Beton u. Eisen* (Germany), Nov. 20, 1930, V. 29, Heft 22, p. 404-406.—Construction of office building for "Berg und Huettengesellschaft" in Prague continued through unusually cold winter of 1928. Building has a ground area of 26,900 sq. ft. with three sub-basements and required an excavation varying from 33 ft. to 53 ft. below street grade. Two measures were adopted to insure satisfactory concrete: protection of site with temporary wooden enclosures, and heating of aggregate. Temperature in these temporary sheds was maintained at about + 36° F. by means of stoves, while the outside temperature occasionally dropped to — 15° F. The aggregate was placed in the mixer in frozen condition. A six nozzled, fork-like fitting was connected to the city gas line and the flames directed into the upper part of rotating mixer drum. No cement or water was added until the aggregate had reached a temperature of 98° F. to 107° F. A curve was made giving the time of heating required for aggregate varying from + 14° F. to + 86° F. in temperature on arrival at the mixer. The curve is almost a straight line and calls for a heating period of zero minutes for aggregate at 86° F. and 5½ minutes for aggregate at 14° F. When the aggregate had been heated, flame was shut off and unheated water and cement added. Total time in machine could have been reduced by adding cement to frozen aggregate and allowing mixing to proceed simultaneously with thawing. Increased production could be obtained by using two mixers, one being employed only for heating purposes. Prof. J. Klokner made a study of the effect of the temperature of mix at pouring, on the compressive strengths of test cubes and found that a 41° F. mix developed 8-day strength of 910 lb. per sq. in. and a 68° F. mix 1710 lb. per sq. in. It is indicated, however, that this difference in strength decreases with time. Method of heating aggregate proved very satisfactory and has great advantage of permitting concreting to begin without delay caused by elaborate preliminary work. It is desirable to heat aggregate to highest possible temperature and to lower temperature of temporary enclosures accordingly, within reasonable limits, since the first procedure is simple and cheap while the second is difficult and expensive.—A. A. BRIELMAIER

Build business district of concrete in residential suburb. (See ARCHITECTURAL DESIGN.)

Form details and practice developed for architectural concrete. (See ARCHITECTURAL DESIGN.)

DAMS

Foundation procedure at Owyhee dam. *Eng. News Record*, Jan. 29, 1931, V. 106, No. 5, p. 178.—Foundation problems, complicated by necessity of excavating and concreting a fault zone 190 ft. below stream bed, have been solved at 530-ft. arched-gravity dam being built for the Owyhee project in eastern Oregon, and concrete placing on the main structure is well started.

Completion of the removal of the 38,000 cu. yd. of material from the crevice in the fault zone permitted concreting operations on a full scale, and about 1,000 cu. yd. per shift are being placed in the structure from an 8-cu. yd. bucket handled by overhead cableway. In May, 1930, the 250-ft. open cut 50 ft. deep representing the length of the fault zone lying under the dam was filled with concrete. Five 12 by 12-ft. openings were left to permit excavation of the faulted material below. As excavation was completed in various section along the bottom of crevice, the timbering was removed and space filled with concrete. The granite aggregate is shipped 25 miles to washing, screening, and mixing plant, located about ¼ mile from the dam. The entire plant is designed to produce 1,000 cu. yd. in 8 hr. Proportioning the mix has as its objective production of a finished concrete, which is an economic balance between the qualities of strength, impermeability, and durability, with due regard for desirability of using all coarse aggregate brought to the site. Use of 1 bbl. of cement per cu. yd. gives a mix varying from 1:2.7:6.45 to 1:2.5:6.65, and a water-cement ratio of 0.9. The time of mixing is 1½ min. The 28-day strength averages 3,500 lb. per sq. in. Concrete pours are in 4-ft. lifts between radial construction joints. A period of 72 hr. is required between pours on the same section. Bond is insured by washing and cleaning the surface between pours. Provision is made for grouting the construction joints later under a pressure of 100 lb. per sq. in.—GLENN MURPHY

The Chute a Caron hydro-electric development. JAMES W. RICKEY. *Proc. Eng. Soc. Western Pennsylvania*, Jan., 1931, V. 47, No. 1, 43 p.—Details of power developments in Province of Quebec with description of drainage basin are given. Methods for concrete construction have been very completely described by I. E. Burks (cf. JOURNAL of the Am. Con. Inst., Feb., 1930, *Proceedings*, V. 26, p. 315). Of outstanding interest was use of precast obelisk of heavy reinforced concrete for diverting river. While work was going on a large reinforced concrete mass was built on the mid-channel island. Special provisions were made in its pedestal to allow blasting away one of supports to cause obelisk to tip into and block the main channel, thus diverting entire flow up to 50,000 sec. ft. into completed diversion channel and thus permitting construction to proceed on main dam in old river bed. The problem included many unknown factors, such as best way to start obelisk falling, effect of impact against surface of water, possible destruction when obelisk landed on river bed, etc. Extensive mathematical analyses were first made and a model built in hydraulic laboratory of Carnegie Institute of Technology on scale of 1 to 50 to represent section of Saguenay River, diversion channel and obelisk. By means of special recording devices, all mathematical analyses were checked and effect of water action in deflecting and cushioning obelisk were determined in the laboratory. Work done in laboratory was repeated in actual construction and data are given to show that prediction in laboratory was fulfilled when obelisk was upset into river so closely that the final resting place was within ¼ in. of pre-determined location.—P. J. FREEMAN

Zschopau dam near Kriebstein (Germany), a cast concrete job. SUTTER. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 467-78.—Zschopau dam, completed in 1929, is concrete gravity dam with total height of 108.3 ft. It is arched on a 738 ft. radius. Crest is 787.4 ft. long and 13.1 ft. wide; greatest width at base is 72. ft. Spillway is arranged in two sections, each having 4 openings 22.9 ft. wide and 13.1 ft. high. All are provided with gates operated from top of dam. At base of dam water passes through three 8.5-ft. pipes with gates capable of discharging about 400 cu. yd. of water per second. Three turbines with an efficiency of 80.5 per cent produce about 8000 h. p. Expansion joints are provided at 65.6 and 82.0 ft. intervals. Copper sheet and asphalt packing were used to make them watertight. Drainage system consists of vertical drains spaced at 6.6 ft. intervals,

which are connected with horizontal drain tile. Concrete was made of mixture of 440-lb. portland cement and 165-lb. trass, special consideration being given to proper gradation of aggregates. Their composition was: 15 per cent sand (0-0.08 in.), 15 per cent sand (0-0.28 in.), 40 per cent gravel (0.28-1.2 in.) and 30 per cent broken stone (1.2-2.4 in.). Central concrete mixing plant was built on left side of river and aerial cableway of 1 mile transported structural materials to construction site. Four 1-yd. mixers discharged into buckets which delivered concrete to 4 distribution towers. (cf. Kirsten, *Bautechnik* (Germany), March-April, 1930, V. 8, p. 149, 233, 247.)—A. E. BEITLICH

MISCELLANEOUS

New German specifications for reinforced concrete. *Zement* (Germany), Jan. 22, 1931, V. 20, No. 4, p. 84-6.—Changes and improvements in new tentative German specifications for construction of reinforced concrete structures especially emphasize improved structural materials and new experiences in field of concrete composition and working.—A. E. BEITLICH

Precast units in slab construction. R. RALL. *Beton u. Eisen* (Germany), Oct. 20, 1930, V. 29, Heft 20, p. 370-373.—A review of precast slabs, beams and hollow tiles used in Germany today gives details of manufacture for various types. Reinforced roof panel of "asbestos-concrete" is one of the new developments. This panel, either flat or corrugated, is used on spans of about 4 ft. Because of the weight of ordinary concrete, panels are usually precast of light weight concrete and used only for roofs. Types cited are reinforced in one direction and vary in thickness from 2½ to 5 in., with spans up to about 8 ft. Various profiles are in use for precast beams. These are either placed side by side to form a slab, or are spaced up to 2½ ft. apart and slab units placed or poured between them. These beams are available for spans up to 20 and 25 ft. The Visintini truss is one of the best known beams. Several tiles for one-way and two-way slabs are shown. Some standardization of practice in span lengths and concentration on certain types are necessary for further development of this field of precast products.—A. A. BRIELMAIER

Modern construction engines. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 64.—New engines of great efficiency which are capable of handling very large quantities of materials were used for construction of large channel to river Rhine between Basel (Switzerland) and Strassburg (France). Dams are about 150 ft. wide and are built with modern elevating graders which discharge materials directly upon embankment. Areas ranging from 1,800 to 2,400 sq. yds. can be planed in 8 days. Concrete mixer and distribution system travels on same tracks and places 314 to 392 cu. yd. of concrete in 8 hr. Reinforcement is placed by same engine.—A. E. BEITLICH

An overturned 19,000-ton caisson successfully salvaged. *Eng. News Record*, Feb. 12, 1931, V. 106, No. 7.—During the construction of the piers for Mid-Hudson Bridge at Poughkeepsie, N. Y., one of the caissons tilted to 42½ deg. with the horizontal, necessitating considerable amount of labor in righting it. Each of two caissons, the largest on record, consisted of a concrete-filled steel cutting edge and truss system 60 by 136 ft. and 20 ft. deep with semicircular ends. The lower steel sections were fabricated and towed to the site, where they were filled with concrete. During filling and sinking of east caisson, the weakened clay walls suddenly gave way causing the caisson to lean and settle. Steps were immediately taken to prevent further movement by anchoring it to tugs, and by dumping gravel on the weakened clay bank. Caisson was finally righted by excavating material under high edge and applying a pull of 1,500 tons to top edge.—GLENN MURPHY

Concrete plant in Boston designed to eliminate production delays. W. E. TRAUFFER. *Pit and Quarry*, Feb. 11, 1931, V. 21, No. 10, p. 55-57.—Forsyth Street plant of Boston Transit Mixers, Inc., Boston, Mass., built entirely of concrete and steel requires only six men for operation. The com-

pany has another concrete plant at Roslindale, with capacity of 55 cu. yd. per hr. It began operation with a fleet of 10 transit mixers. When Forsyth plant went into operation, 20 of same trucks were added to fleet to serve both plants. Aggregates at new plant are discharged into steel hopper and belt-conveyed to three compartments of a 166-cu. yd. steel bin. Aggregates are conveyed to top of plant where a swivel chute deposits them in four of five compartments concrete bin; the fifth compartment of 250 bbl. capacity, is for cement. Aggregates are sand, two grades of gravel, and one grade of crushed stone. Bulk cement is unloaded from cars and pumped by pneumatic system to two concrete storage silos, from which it is moved, as required, to the 250-bbl. cement compartment by same 5-in. pump. Aggregates are fed through radial gates and cement is fed through a rotary valve into weigh hoppers. Each hopper has a scale with a weightometer attachment. Contents of the two weigh hoppers are discharged to charging hopper which, in turn, discharges mix dry into truck. Water is fed simultaneously to tank on truck. Deliveries are usually within a radius of five miles. Distances of 20 miles are not unusual. Most of market for concrete is for laying sidewalks and highways.—A. J. HOSKIN

Construction of the North Lock in Bremerhaven (Germany). A. AGATZ. *Report 33rd. main meeting German Concrete Association*, March 17, 1930, p. 324-88.—Author gives extensive description of enlargements and new constructions of North Lock. Entire construction program covers: (1) Outer harbor, 1148.3 ft. long, 393.7 ft. wide at harbor entrance and 262.5 ft. wide at lock entrance; depth of water along breakwater is 52.5 ft. (2) Lock, 1220.5 ft. long and 196.8 ft. wide with 47.6 ft. of water and 147.6 ft. wide passage opening. (3) Turning basin, 1312.3 ft. long and 787.4 ft. wide; depth of water is 42.7 ft. (4) Connecting canal, which is 147.6 ft. wide. (5) Swing-bridge, distance between rest piers is 367.2 ft., main girders are placed 53.1 ft. apart and carry two railroad tracks for freight and passenger service, one wide roadway and a 9.2-ft. wide cantilevered sidewalk. (6) Enlargement of inner harbor to a length of 984.3 ft. and a depth of 42.7 ft. (7) Extension of a dock from 879.3 ft. to 1099.1 ft. Work was started in 1927 and will probably be finished in 1931. After its completion, this harbor will be one of greatest European ports and capable of meeting greatest traffic and largest ships. Over 400 holes were drilled to depths of 164 ft. to study underground conditions. Results of this investigation are given in details. Axis of new outer harbor structure is about 154.2 ft. farther west than axis of old harbor. Construction of break-water and harbor walls was carried out in great number of divisions for which 20 different cross-sections were calculated according to variations of underground. Concrete walls rest on wooden pile foundations, which were more resistant than concrete piles in this particular kind of sea water. About 600-lbs. blast furnace slag cement and 100-lbs. trass were used for 35.3 cu. ft. of concrete. Top of walls was provided with 20-in. wide strip of basalt slabs roughness of which facilitates walking. Steel sheet pilings of Larrsen type were used for wall construction. Inner and outer head of lock are of similar design and were built under considerable difficulties due to bad underground. Construction was of reinforced concrete after lowering groundwater level. Bottom slab is 21.3 ft. thick. Water pipes are shaped like trumpets in order to furnish favorable flow conditions. Lock can be filled in 18 to 29 minutes. Shape and reinforcing of heads are illustrated in several drawings. Smallest width is 32.8 ft. greatest 39.7 ft. and height is 65.6 ft. Gates are of sliding type, 27.9 ft. wide, 62.3 ft. high and 152.9 ft. long. Steel structure of swing-bridge rests on heavy reinforced concrete foundation constructed between steel sheet pilings on sound underground. Two end piers rest on wooden pile foundations. Three engine houses were erected for operation of gates and swing bridge. It is also necessary to build a 220 ft. extension to old dock, which is now 879.3 ft. long. Ten different cross-sections are calculated and discussed. New dock will be able to hold ships with capacities of 80,000 tons. Last part of article gives data about amount of consumed structural materials,

volume of excavations, arrangement of site of construction, progress of construction program and general information. (cf. *Bautechnik* (Germany), June 6, 1930, V. 8, No. 25.)—A. E. BEITLICH

Precast concrete balconies. *Conc. Building Conc. Products* (England), Dec., 1930, V. 5, No. 12, p. 162.—An illustration is given of a new type of precast concrete balcony for use in cinemas and theatres. The treads and risers are precast in one unit with continuous reinforcement, and at the back of tread on underside, is a notch to fit rolled steel joist supporting the stepping. On top side is another notch to receive the riser above. The risers are 2½ in. and the tread 3½ in. thick. The edges are formed to make joggled joints, and strips of wood are cast in the treads for nailing floor boards, etc.—JOHN E. ADAMS

Omaha firm sells both central-mixed and mixed-in-transit concrete. *Pit and Quarry,* Nov. 19, 1930, V. 21, No. 4, p. 40-43.—Ready Mixed Concrete Co., Omaha, Neb., has new central plant to produce concrete by two methods. Aggregates and cement flow by gravity from respective hoppers into weighing batchers and then to truck-loading chute. The aggregates hopper has two compartments of combined capacity of 75 cu. yd. Cement hopper holds 20 tons. Cement's flow to batcher is controlled by a dumping gate. A timing dial on aggregates scale indicates approach of correct weight. Batch hopper has a capacity of 10 tons. The cement batcher has a capacity of 2,500 lb. and has a similar scale. A small 110-volt vibrator is mounted on cement-discharge to prevent cement from sticking. Water is measured volumetrically from a square steel tank by a short length of pipe inside the tank capable of being raised or lowered to vary the quantity of water withdrawn in 5-gal. units, up to 120 gal. "Sand-gravel" in this field means mixture all of which will pass 1-in. mesh, and 50 per cent of which will not pass a No. 10 screen. This aggregate is received in carloads at plant, and is handled directly from storage into hoppers. Cement, purchased in bulk, unloaded from cars by power shovel into small receiving hopper at ground level. Thence it passes to the cement hopper or to storage. As needed stored cement is fed again to the bucket elevator. Delivery of concrete is made free of charge within Omaha city limits, with a charge of 25¢ per cu. yd. mi. elsewhere. Four mixes are standardized and sold under guaranteed strengths varying from 2,000 to 3,600 lb. per sq. in. An electric system of communication enables the shipping clerk in his office to flash a series of numerals at the batching operator's station to indicate the quantity of an order and the mix.—A. J. HOSKIN

Concrete bulkheads for creek channel improve Jacksonville swamp area. (See ENGINEERING DESIGN—MISCELLANEOUS.)

Concrete block tunnel lining. (See SHOP MANUFACTURE.)

Reinforced concrete window frames. (See SHOP MANUFACTURE.)

Some new prospects in the development of the concrete products industry. (See SHOP MANUFACTURE.)

ROADS AND PAVEMENTS

Rigid inspection produces better pavements. *Eng. News Record,* Jan. 8, 1931, V. 105, No. 2, p. 77.—Supervision of contract paving in Detroit by city engineer's office during last 6½ years has increased the 28-day strength of drilled cores by 25 per cent. By making richer mix, eliminating soft, dusty limestone aggregate, and increasing the time of mixing, the average 28-day compressive strength of drilled cores has been increased to more than 3200 lb. per sq. in.—GLENN MURPHY

Truck operation and production in concrete paving work. A. P. ANDERSON. *Public Roads*, Feb., 1931.—Analyses of time losses in production suffered by reason of either an insufficient supply or faulty operation of hauling equipment used in concrete road construction, and results of studies on 100 concrete paving jobs show an average time loss of 17 per cent due to these causes. These time losses may be considerably reduced by applying a number of governing principles which are discussed in detail in the report indicating importance of properly designing the material yard layout and shows the amount of loss due to faulty layout. Tables and charts are included from which the proper number of trucks to use under any given set of conditions may be computed. Contract hauling is not in general profitable to contractor, because conditions are often such that greatest profit to hauling contractor accrues at rather low rates of production.—F. H. JACKSON

A study of methods of curing concrete pavements. F. H. JACKSON AND E. W. BAUMAN. *Public Roads*, Jan., 1931.—In field investigation in Tennessee conducted jointly by the Bureau of Public Roads and Tennessee Department of Highways, experiments were conducted on a concrete pavement approximately 15 miles long, pavement on one side of longitudinal joint being cured in 1000-ft. sections with the various curing methods under consideration while concrete on other side of joint was cured with earth and water in the conventional manner for comparison purposes. The following methods of curing were investigated: Wet burlap applied for 12 to 96 hrs.; no cure; Sisalkraft paper applied for 24 hrs.; sodium silicate—earth subgrade, tar paper on subgrade; calcium chloride, surface application—earth subgrade, tar paper on subgrade; Hunt process—earth subgrade, tar paper on subgrade; Barber asphalt emulsion (curcrete)—earth subgrade, tar paper on subgrade; Headley asphalt emulsion—earth subgrade, tar paper on subgrade; Tarvia K. P.—earth subgrade, tar paper on subgrade; Tarvia B—earth subgrade, tar paper on subgrade; poor earth cure; ponding; calcium chloride admixture—earth subgrade, tar paper on subgrade. Total length of pavement tested was 74,148 ft. A large number of concrete beams and cylinders were made and tested and results analyzed in detail. In addition results of two detailed crack surveys of pavement are given. Conclusions as to relative efficiency of various curing methods are based on consideration of strength developed by concrete beams cast alongside pavement sections and cured in same manner, concrete cores drilled from the pavements and the results of crack surveys. Efficiency of curing method as applied to each section was determined by comparing results for that particular section with results obtained for the standard water cure on the same section. In this way accidental variables such as subgrade conditions, etc., were largely eliminated. It is believed that results of this investigation justify following comments relative to merits of various curing methods as compared with standard method of curing with earth and water, under conditions prevailing during progress of these tests: 1. *Burlap curing*—Method of cure involving application of wet burlap for periods varying from 24 to 96 hr. without further curing compares quite favorably with standard method. No final conclusions can be drawn until additional tests under more severe weather conditions are made. 2. *Sisalkraft paper*—Application of Sisalkraft paper for periods of 24 to 48 hr. without further curing is a reasonably satisfactory method of cure compared with standard method. 3. *Poor earth cure*—It is not necessary to keep earth saturated with water for entire 10-day curing period. 4. *Sodium silicate*—This method of cure gives results somewhat less satisfactory than those given by standard method. 5. *Calcium chloride, surface application*—Results very nearly as satisfactory as standard method. 6. *Calcium chloride admixture*—Sections on which admixture was used are too short to warrant any conclusion as to merit of this method. 7. *Bituminous materials*—Curing with bituminous materials, as used in these tests, is unsatisfactory, because of the marked evidence of increased transverse cracking. 8. *Tar paper on subgrade*—Nothing is gained by its use.—F. H. JACKSON

SHOP MANUFACTURE

To prevent cracks in masonry walls. W. D. M. ALLAN. *Concrete Products*, Jan., 1931, V. 40, No. 1, p. 27-29.—This article was prepared to provide products manufacturers with information to refute the claims of competing materials that cracks necessarily occur in concrete masonry. A number of drawings illustrate the best methods of preventing cracks.—E. S. HANSON

Long established pipe plant adopts machine manufacture. *Concrete Products*, Feb., 1931, V. 49, No. 2, p. 20-23.—The Core Joint Concrete Pipe Co., at Irvington, N. J., which has been manufacturing hand made pipe for a number of years, has recently added a department for making pipe by machine, the process being of particular interest in that it provides for lining the pipe by machine at the same time as it is manufactured. A mix is applied hot and automatically trowelled into the wall of the freshly made pipe. This liner mix is composed principally of specially graded sand, slate flour and ordinary paving asphalt.—E. S. HANSON

Das betonwerkstein (cast stone) TAMMS. *Betonwerk* (Germany), Dec. 21, 1930, V. 18, No. 51, p. 715.—In present methods of making cast stone discussed, aggregates are selected for their durability and appearance, carefully graded and recombined. The surface of the cast product is cut to resemble cut stone. It is suggested that this method may not be ideal but a surface which reflects the mold in which it is cast may produce a product more in keeping with the nature of concrete. Under certain conditions such as with the use of a hard aggregate the employment of the sand blast may produce interesting surfaces, also the Contex method is suggested in that individual grains or aggregates are fully exposed. Examples in France and America are cited, especially, several examples of work done by Frank Lloyd Wright in California.—H. FRAUENFELDER

A new column fireproofing unit. ROBT. H. MOORE. *Concrete Products*, Jan., 1931, V. 40, No. 1, p. 22-23.—Segmental units for fireproofing steel columns were developed by the Detroit Cinder Block & Tile Co., Detroit, Mich., for use on one particular job but have proven so successful as to be added to the regular line of products. These are made in circular and octagonal shapes, the circular units three to a circle and the octagonal two to a circle. The units are 2 in. thick and 8 in. high, with the length depending on the size of the column to be fireproofed.—E. S. HANSON

Profitable manufacture of concrete building units. FRED A. SAGER. *Concrete*, March, 1931, V. 38, No. 3, p. 27-29.—With approximate idea of space requirements of each part of manufacturing process, preliminary block plan may be drawn up. Use of outline sketches of various parts of plant, drawn to same scale as property site and cut out on separate pieces of paper, to be arranged in various combinations so that advantages of different locations may be studied, is suggested. Two fundamental characteristics of layout should be arrangement for future extension, and one-direction flow lines for material through the plant. Equipment space includes area for elevator and crusher adjacent to cinder storage and for boiler near coal storage, all located near railroad siding. Manufacturing plant includes two standard block machines, brick machine, hand-operated machine, mixer, motors, storage for pallets, etc. Advantageous stock-piling and area required are determined. Diagram illustrates provisions for materials, equipment and traffic.—C. BACHMANN

New product gave us our best year. C. V. BERRY. *Concrete Products*, Feb., 1931, V. 49, No. 2, p. 17-19.—Addition of manufacture of concrete floor

joists gave a products manufacturer in Kalamazoo, Mich., a successful business during 1930 in spite of unfavorable conditions. These floor joists are shaped somewhat like a steel I-beam, having top and bottom flanges 3 in. wide and a web thickness of 1¼ in. The section is made 8 or 10 in. high and up to 20 ft. in length. Adequate tensile and sheer reinforcement is provided in a welded unit to resist externally imposed loads with an ample factor of safety. Sand and Haydite are used as aggregates, the mixture being proportioned as 1 part cement, 2 sand, 2 fine Haydite and 4 coarse Haydite. Special equipment has been developed for manufacturing these floor joists approximating somewhat the method of rolling steel rails in a steel mill—E. S. HANSON

Concrete sign posts. *Conc. Bldg. Conc. Prod.* (England), Feb., 1931, V. 6, No. 2, p. 18.—Precast concrete sign post, made with colored cements, results in saving of maintenance costs. Posts could be in white cement, and the arms white with black letters or vice versa. Cap could be of any standard design. Letters are cast separately and attached to arms later, arms being left with a rough face. Rough surface may be formed by fixing ½-in. mesh wire netting to the panel which forms sinking for letters, so when the side forms are released netting comes away also, leaving a surface with a good key. Letters should be ⅝ in. in thickness with ⅜ in. embedded in concrete, and composed of 1:2 mix with clean sharp sand. Fixing is done with a slight skimming of neat cement and filling made with 1:3 mix in the desired color. The suggested mixes are as follows: 1:1½:3 (½ in. to ⅛-in. ballast with clean sand or stone dust) for posts; 1:1½:3 (⅜ in. to ⅛ in. as above) for arms and caps. Reinforcement for post should be four ½-in. rods with ⅛-in. wire ties at 12-in. centres, and with 1-in. cover. For arms two ⅜-in. rods tied at 6-in. centres are recommended.—JOHN E. ADAMS

Concrete block tunnel lining. ERNEST H. ROWE. *Concrete Products*, Jan., 1931, V. 49, No. 1, p. 12-14.—Manufacture and installation of segmental concrete block for lining one of extensions of New York's system of rapid transit subways include interesting details. The bid of the successful contractor was $387.00 per lineal ft. of lining as against $524.00 per lineal ft. for cast iron lining. In addition to the money saving the concrete block tunnels were completed in much less time than cast iron tunnels. Ten blocks per ring were specified, outside diameter of ring to be 3 ft. greater than inner diameter which was 16 ft. 9 in. Each block was 5 ft. 2⅞ in. on the arc of the concave face with a width of 2 ft. 6 in. and a radial thickness of 18 in. Forms for each block were set up on a car and the cars handled in trains of 30 under mixer. Each block was reinforced with two mats of five ⅜-in. rods. Interlocking of the rings was provided for by two projections, on one side, and corresponding depression on opposite side.—E. S. HANSON

Reinforced concrete window frames. W. CHRISTEN. *Beton u. Eisen* (Germany), Oct. 20, 1930, V. 29, Heft 20, p. 375-377.—The speed with which modern structures are being erected has created a demand for factory-built and standarized building units. These units lead to an improvement of the construction and to lower costs. Window openings had formerly been built up on the job of separate precast units or of brick. Necessity of economizing on labor has led to quantity production of reinforced concrete window frames—a product which has come into greater use in the last few years. Advantages of this ready-built unit are: more effective and exact construction; lighter weight (a frame unit with 4 ft. 7 in. by 3 ft. 9 in. opening weights 310 lb. as against 1210 lb. formerly necessary); easier erection. In order to further the use of this type of unit it is necessary to standardize openings and frame sections. Experience has shown that less than 20 pieces of one type cannot be manufactured at a price low enough to compete with the older methods. (Abstractor's note: In accordance with German practice these frames are all for casement windows.)—A. A. BRIELMAIER

Media plant pioneers in manufacture of light-walled units. J. J. BUZZELL. *Concrete*, March, 1931, V. 38, No. 3, p. 21-22.—Media Concrete Products Co., Inc., Media, Pa., has successfully developed light-weight building unit of improved design, using ordinary aggregates. In addition to old style standard sizes, units 7¾ by 7¾ by 15¾ in. and 7¾ by 11¾ by 15¾ in. have been adopted. Fifty per cent air space was found most satisfactory. Mortar bed of 1½ in. is provided due to sharp flare at top ledge. Units are manufactured in two classes. First employs combination of hard aggregates resulting in dense concrete, not nailable but intended for underground work. 7¾ by 7¾ by 15¾ in. size weighs 36 lbs., and 7¾ by 11¾ by 15¾ in. size weighs 45 lb. Other type of light-walled unit provides extreme light-weight and free nailability with slight crooking of nail. 7¾ by 7¾ by 15¾ in. size weighs 30 lb. and 7¾ by 11¾ by 15¾ in. size weighs 38 lb. Besser senior stripper and 25-ft. mixer, and one size plain pallets for all units are used. Air-tight steam enclosures at siding are provided for thawing out cars of freezing aggregates. Large block storage area is at car level adjacent to siding, so cars may be loaded by rolling units from stock-piles into cars. Overhead storage capacity is divided into nine separate, covered bins equipped with steam heating facilities. Mixer floor is directly beneath overhead bins, mixer hoppers are fed from any bin desired by 18-in. cross conveyor and machines are located directly under mixers on lower floor. Arrangement of racks around machines allows continuous machine operation, using electric industrial track.—C. BACHMANN

Some new prospects in the development of the concrete product industry. R. JECHT. *Beton u. Eisen* (Germany), Oct. 20, 1930, V. 29, Heft 20, p. 368-370.—The profitable manufacture of concrete units depends on two conditions; a large or steady demand for same product, need or economy of high-quality product for special purpose. The first condition has led to mass production of pipes, slabs, poles, etc. In some cases the same product satisfies both conditions. Class of units meeting second condition is not clearly defined. A high-quality product requires long experience, expensive experimental work and thoroughly trained workmen. Best results of progress made in concrete proportioning and treating can be realized only in permanent factory with experienced personnel. Progress in widening the field is marked by costly failures. Aim to manufacture units which will utilize, on the job, modern transportation and erection equipment is correct. Only in this manner can concrete products benefit, as steel has done, from progress made in erection equipment. A few examples follow: Highway viaduct in Klingenberg, Saxony, is built of A—frames with centrifugally-cast poles up to 56 ft. long. Frames are stiffened longitudinally by the deck and at about one third height by two horizontal members from frame to frame. The corners and connections were made secure after erection by pouring mortar in the joints or casting a concrete block around them. The form costs for pouring this job on the site would have been prohibitive. Units for poles and towers for pipe lines and high tension lines have come into use but face a very strong competitor in steel, due to the present low steel prices. In Italy where the position of steel is not so strong, these towers have been more widely used. Recently in the improvement of Konigsberg Sea canal 60 three-pile (centrif-cast) foundations were used for sea beacon bases. These piles are exposed to a strong ice pressure. Use of precast units, as wall panels, roof slabs, has proved economical for garages and other small jobs. In Germany the use of concrete water pipes is just becoming noticeable while in other countries much progress has already been made. Here again the centrifugally-cast product is in the lead.—A. A. BRIELMAIER

Technique and form in cast stone. (See ARCHITECTURAL DESIGN.)

Precast units in slab construction. (See FIELD CONSTRUCTION—Miscellaneous.)

Materials

Admixtures

Slaked lime as a means towards denser concrete. R. Grun. *Summary Reports First I. C. for C. and R. C.*—Relations between impermeability and strength of concrete: Against physical effects; against chemical action. The making of watertight concrete: Effect of lime upon: plasticity, tensile and compressive strength, compactedness, resistance to frost, resistance to chemical action.—La Technique des Travaux

Effect of several admixtures on portland cement. P. P. Budnikoff and M. I. Nekritsch. *Zement* (Germany), Feb. 26, 1931, V. 20, No. 9, p. 192-3.—In experiments at laboratory of technology of silicates in Charkow (Russia), cement was mixed with 1, 2, 5, 10 and 30 per cent of each of following materials: chalk, dolomite, dolomite burned at 750° F., dolomite burned at 1470° F., marl and marl burned at 1110° F. Admixtures were ground together with clinker in laboratory ball mill. Standard 1:3 mortar specimens were made for compressive and tensile strength tests and stored in water and artificially made sea water. Tensile strength of pure cement is in general slightly higher than that of cements with admixtures. Compressive strength is mostly improved by above materials. Admixtures increase density and adsorbing properties of mixtures. Additions of as much as 10 per cent do not affect value of cement. Most economic admixture is chalk. Large scale tests along same lines are in progress.—A. E. Beitlich

Aggregates

The composition, mixing, methods of application of concrete and reinforced concrete as used in the yards and the control of same. T. R. Grigson. *Summary Reports First I. C. for C. and R. C.*—The paper is divided into various headings which may be summarized as follows: Composition of concrete; cement, aggregates, proportioning, admixtures and fillers. Mixing concrete; methods of application.—La Technique des Travaux

The application of sieve analyses and Abrams' fineness modulus. A. Hummel. *Summary Reports First I. C. for C. and R. C.*—Influence of gauge of inert materials on strength of mortar and concrete has been zealously studied in laboratories during past 25 years. Results obtained have been correlated in ideal granimetric curves (Fuller, etc.) or rules for gauging. The value of these curves for scientific research and for laboratory practice is unquestionable, but for use on the works it is questioned. This paper indicates a more general method of estimating qualities of inert materials from grainmetric point of view.—La Technique des Travaux

Criticism of fineness modulus. A. Hummel. *Zement* (Germany), Feb. 5, 1931, V. 20, No. 6, p. 129-34.—Author discusses investigators' comments on his paper "The valuation of sieve analyses and the fineness modulus by Abrams" *Zement* (Germany), April 1930, V. 19, No. 15, p. 355-64; Journal A. C. I., June 1930, V. 1, No. 8, *Abstr.* p. 108. Too refined mathematical calculation can not be applied to this subject without making method valueless for practice. Advantages and disadvantages of method, especially its speed of calculation, are illustrated.—A. E. Beitlich

Ideal curve, fineness modulus and the new standard specifications. Kristen. *Zement* (Germany), April 16, 1931, V. 20, No. 16, p. 366-9.—A comparison is made of ideal curve, curve by Otzen and Abrams' fineness modulus. New text of German standard specifications for reinforced concrete is discussed. Best and most economic way to improve a material is to compare its sieve analysis with ideal curve (without cement). If it is necessary to add coarse materials, percentages of these additions can be found with fineness modulus by method of Hummel. Graphical method for plotting curves is explained, especially conversion of Tyler sieve openings into metric equivalents. —A. E. Beitlich

About mesh openings and particle sizes. JOHS. ANDERSEN. *Zement* (Germany), March 5, 12, 1931, V. 20, No. 10-11, p. 224-6, 242-5.—Study was undertaken to replace measurement of mesh openings in sieve analysis by determination of average diameter of particles which can just pass sieve when sieving process is completed. This fraction is called size separation (Kornscheide). Investigation covered (1) pure separation, (2) relation between average mesh opening and size separation of certain sieves, (3) demonstration of sieve analyses and (4) size separation of sieves with different materials. Fineness curves are much smoother than those based on mesh openings. Relation of size separation to mesh opening is about 0.81 in case of mostly spherical particles and slightly less for particles with sharp edges.—A. E. BEITLICH

Synthoporit—A new light-weight structural material. H. F. KRAUSE. *Zement* (Germany), April 16, 1931, V. 20, No. 16, p. 372-4.—A very porous material is prepared from a fusion of calcium silicate. Melt leaves furnace at 2730 to 2910° F. and is chilled by patented process. Cold slag is then broken and sieve separated into certain fractions. Material consists of about 20 per cent solids and 80 per cent voids. It floats on water even after 3 months. It is not affected by high temperatures and its strength is not materially decreased when used as concrete aggregate and exposed to high temperatures. Chemical composition is close to wollastonite, 47-49 per cent SiO_2, 2-4 per cent Al_2O_3, 0.2-0.5 per cent FeO and 43-45 per cent CaO. Sulphur is below 0.3 per cent. Specific gravity is 0.6. Voids are mostly closed and do not absorb much water. It is highly heat insulating. Greatest field of application is concrete. It is used for manufacture of slabs, light weight bricks, ceilings, encasings for steel structures and walls. Concrete with synthoporit as aggregate has very favorable strength properties.—A. E. BEITLICH

Gravel plant of unusual design. H. M. FITCH. *Rock Products*, March 14, 1931. V. 34, No. 6, p. 33.—Hillside Washed Sand and Gravel Co.'s plant at Muskego, Wis., has screening and washing machinery mounted on an octagonal tower. Walls, or fins, run out from each angle of tower making eight spaces. Conveyor for recovering materials is installed in one space leaving seven for storage. Boot of conveyor is in base of tower and chutes from all divisions lead to it. A hoist is moved on a circular track around the storage to handle a scraper bucket to pull material in and out of storage spaces.—EDMUND SHAW

Quarry practice at the operations of the John T. Dyer Quarry Co. *Rock Products*, Feb. 28, 1931, V. 34, No. 5, p. 47.—The John T. Dyer Quarry Co., which has three plants near Birdsboro, Penn., is developing blasting technique and solving the problem of making the fines into a commercial product. Coyote holes are used at the Monocacy Plant. They are driven from a tunnel about 100 ft. back of the face, this tunnel being connected to the surface by a vertical shaft. The face is 300 ft. high and two tunnels will break the rock in two benches. At the Clingan Quarry well drill holes are set 20 ft. apart with 35 ft. burden. The rock broken ranges from 6 to 7 tons per lb. of powder. Four ton auto car trucks haul the rock to the crusher. The main crusher of the Monocacy Plant (60 by 84 in.) was once the largest jaw crusher in the world. Drive for this is most unusual. Fines have been sold as fine aggregate and recently part of them have been ground very fine to be used in the recovery of fine coal.—EDMUND SHAW

Proposed method of test for soundness of fine and coarse aggregate by use of sodium sulphate. Appendix 2 to the fifth annual report of the engineering and research division of the National Sand and Gravel Association. *Nat. Sand Gravel Bul.*, Feb., 1931, V. 12, No. 2, p. 41-42.—A procedure to be followed in treating fine and coarse aggregates to determine their resistance to disintegration by a standard solution of sodium sulphate, and equipment to be used. The solution, prepared by dissolving sodium sulphate in water at about 86° F., is allowed to cool to 70° F. and stand at least 12 hr. before use. For fine aggregate samples a balance of not less than 1,000 grams, sensitive to at least 0.5 gram, is required. Fine aggregate

passing ⅜-in. sieve shall be of such size that approximately 500 grams is coarser than No. 50; that portion finer than No. 50 shall be discarded. Fractions retained between No. 50 and 30, 30 and 16, 16 and 8, 8 and 4, and coarser than No. 4 sieves shall be weighed and placed in separate containers. For coarse aggregate samples a balance of not less than 5,000 grams, sensitive to at least 1 gram, is required. Coarse aggregate shall consist of material from which sizes finer than No. 4 sieve have been removed. Pieces showing incipient fracture due to crushing shall be removed. Weight of sample shall depend on maximum size of aggregate. Fractions retained on ⅜, ¾ and 1½ in. (square openings) shall be weighed and placed in separate containers. Number of particles shall be counted in fraction coarser than ¾ in. Immersed samples shall be stored at 70° F. for a period of 18 hr., and dried in an oven (from 212 to 230° F.) to a constant weight (not less than 4 hr.) then allowed to cool to room temperature. Care must be exercised to avoid loss of aggregate particles or detritus coarser than 100 mesh. Alternate immersion and drying shall be repeated until five cycles (unless otherwise specified) are obtained. Fractions coarser than ¾ in. shall be examined qualitatively after each cycle and quantitatively after completion of test. Fractions finer than ¾ in. shall be examined quantitatively only and after completion of test.—P. McKim

Aggregates and the physical properties of concretes: their coefficient of elasticity and their resistance to compression, tension, flexure, shear and impact. R. Dutron. *Rev. materiaux construction trav. publics* (France), July, Aug., Sept., Oct., Nov., Dec., 1930, No. 250, p. 258-62; No. 251, p. 293-8; No. 252, p. 343-9; No. 253, p. 383-9; No. 254, p. 431-5; No. 255, p. 473-7.—The coarse aggregate was Rhine or Meuse pebbles, or crushed porphyry, slag or sandstone; fine aggregate was Rhine sand or finely crushed porphyry. Some experiments were made with fine slags also. The best compressive strengths were obtained with mixtures of sand and gravel, because of the better packing so that less water was required. Three gradings were used with three different amounts of cement at three different consistencies each. Two kinds of cement were used. The concrete made of river sand and gravel is stronger under compression than when made of crushed rock. Deformability of concrete is influenced by the grading of sand and by the ratio of fine to coarse. Tensile results generally reversed those of compression, porphyry giving better values than gravel and fine crushed rock better than sand. Ratio of tensile to compressive decreases parabolically with age and for crushed stones lies in the range 0.093 to 0.075, while with water worn aggregates, 0.070 to 0.052. The flexural strength equals 1.63 times tensile + 12.5 per cent. Shearing resistance was determined by making beams with an offset center and was found to be closely proportional to compressive strength. Resistance to impact was determined. Specimens of crushed aggregate withstood pounding better than water worn materials. Mixtures gave intermediate results.—F. O. Anderegg

Cement

The cause of the "sanding off" of high alumina cement. A. F. Roscher-Lund. *Zement* (Germany), Feb. 19, 1931, V. 20, No. 8, p. 164.—Softening and sanding off of layers of pats of certain high alumina cements is due to too rapid drying out of surface causing incomplete hydration of cement. Carbonation which has been found to take place in these layers is only of secondary nature and not directly responsible for sanding effect. Opinion of H. W. Gonell on this subject is discussed.—A. E. Beitlich

The cause of setting failures of high alumina cement. H. W. Gonell. *Zement* (Germany), Feb. 19, 1931, V. 20, No. 8, p. 164-8.—Contrary to findings by Roscher-Lund, (cf. *Zement* (Germany), Feb. 19, 1931, V. 20, No. 8, p. 164), carbon dioxide and not too rapid drying is cause of sanding off of high alumina cement test pats. Tests carried out in previous years were repeated. Specimens were stored in moist air, moist carbon dioxide atmos-

phere, moist oxygen and dry oxygen. Same results were obtained. Differences in findings are due to different lime contents of investigated cements. Drying out of surface layer is only of minor importance.—A. E. BEITLICH

Notes on the use of special cements on the Netherlands East Indies state railways. P. A. JELLEMA. *Summary Reports First I. C. for C. and R. C.*—Rapid hardening cements of French, German and British manufacture have been used and in no case were strengths claimed for them obtained. It was established that capacity for rapid hardening diminished with the lapse of time. Control tests indicate that for the present these special cements should, therefore, only be adopted with the greatest care. The reason why the special cements deteriorate in quality is not yet known.—LA TECHNIQUE DES TRAVAUX

Improvements increase output of Dewey cement plant at Davenport, Ia. *Pit and Quarry*, April 8, 1931, V. 22, No. 1, p. 49-55, 61.—Plant which has been using closed-circuit wet finish grinding for about one year, first went into operation in 1927 but numerous improvenents were made prior to 1930, such as development of a new quarry level and a new clay pit, addition of several locomotives, enlargement of the crushing plant, installation of equipment for sizing aggregate for commercial trade, a clay-wash mill, a third kiln and a cooler, a wasteheat boiler, 24 storage silos, and slurry filters.—A. J. HOSKIN

Determination of magnesium in portland cement and similar materials by the use of 8-hydroxyquinoline. J. C. REDMOND AND H. A. BRIGHT. *Bur. of Standards Journ. Research*, Jan. 1931, V. 6, No. 1, p. 113-20.—Procedure given for determination of magnesium in portland cement is accurate and much more rapid than standard phosphate method. Directions for preparation of solutions are given. Determination can be carried out in about 2 hrs.—A. E. BEITLICH

Contribution to the ball mill theory. W. BARTH. *Zement* (Germany), April 23, 1931, V. 20, No. 17, p. 390-1—Reply to article by Naske (cf. *Zement*) (Germany), Feb. 12, 1931, V. 20, No. 7, p. 148-9) in which author states that materials in ball mills are only crushed by impact and not also by attrition. He explains ways of particles in mill and theory by Fischer. Naske gives his final opinion which is contrary to conception of author.—A. E. BEITLICH

About the manufacture of fused cement. WALTER MARSCHNER. *Zement* (Germany), April 16, 1931, V. 20, No. 16, p. 362-3.—Improvement of electric furnace for manufacture of high alumina fused cement facilitates discharging of materials. Temperature of melt must be raised considerably at point of discharge. This can be done economically by adding about 20 per cent of raw mixture to melt at outlet of furnace. Exothermic reaction of chemical constituents in mix causes rise in temperature which is sufficiently high for discharge. German patent (P 58,489) has been applied for.—A. E. BEITLICH

The degree of lime saturation by Kuhl. M. SPINDEL. *Zement* (Germany), April 9, 1931, V. 20, No. 15, p. 338-40.—A contribution to the question of final form of degree of lime saturation derived by Hans Kühl and modified by Hess (cf. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 28-32; JOURNAL A. C. I., April, 1931, V. 2, No. 8, *Abstracts*, p. 214). Author demonstrates degree of lime saturation in his four-component parallelogram and suggests further changes of this modulus. Application in practice is discussed. Kühl, in addition to his previous paper (cf. *Zement* (Germany), Feb. 5, 1931, V. 20, No. 6, p. 123-5), criticizes efforts by Hess and Spindel to modify his formula and claims that "degree of lime saturation" is only to be used for his original modulus.—A. E. BEITLICH

The degree of lime saturation by Kuhl. M. SPINDEL. *Zement* (Germany), April 16, 1931, V. 20, No. 16, p. 360-2.—Continuation of polemic between Spindel and Kühl (cf. *Zement* (Germany), April 9, 1931, V. 20, No. 15, p. 338-40). Author claims that calculation of his formula and of Hess' number can be simplified greatly by mathematical manipulation. Kühl gives short

reply and states that practice must decide which method is most suitable for proportioning of raw materials.—A. E. BEITLICH

The formation of minerals from melts of kaolin-lime mixtures. I. WEYER. *Zement* (Germany), March 19, 1931, V. 20, No. 12, p. 264-5.—Mixtures of kaolin and lime were heated above sintering point and formation of minerals investigated to determine whether tricalcium silicate, already formed at this temperature, undergoes any changes. Five different brands of metakaolin and lime in proportions 1:8, 1:9, 1:10 and 1:12 were heated from 5 to 6 hr. at 2640° F. Mostly dicalcium silicate in beta form was observed in mixtures with less than 9 mols CaO. In mixtures with more than 9 mols CaO no dicalcium silicate was observed; all silica is combined as tricalcium silicate. —A. E. BEITLICH

The lime content of high alumina cement. HANS KUHL AND SETSUO IDETA. *Zement* (Germany), March 19, 1931, V. 20, No. 12, p. 261-4.—Efforts were made to determine most favorable lime content for high alumina cements by preparing several synthetic melts and studying their physical properties by small-piece testing method. Ratios of SiO_2:Al_2O_3:Fe_2O_3 were 15:35:10, 10:40:10 and 5:45:10. Lime contents in these cements ranged from high to low limit, calculated by means of formula for degree of lime saturation derived by Kuhl (cf. *Tonindustrie Zeitung* (Germany), March 20, 1930, V. 59, No. 23, p. 389-92). No direct relation was found between physical properties and chemical composition.—A. E. BEITLICH

Analysis of standard cements. H. BURCHARTZ. *Zement* (Germany), March 19-26, 1931, V. 20, No. 12-13, p. 258-60, 290-2.—A method has been worked out under supervision of Burchartz, Gruen, Guttmann, Haegermann and Luftschitz for analysis of portland cement, iron portland cement and blast furnace cement. Directions are given for preparation of samples, solutions and their standardization. Analysis includes determination of loss on ignition, silica, sesquioxides of iron and aluminum (special method is given for samples containing more than 1.5 per cent manganese), lime, magnesia, insoluble residue and sulfate.—A. E. BEITLICH

New ways in American cement manufacture. CARL NASKE. *Zement* (Germany), March 19-26, 1931, V. 20, No. 12-13, p. 266-8, 292-6.—Review of several articles gives special consideration to installations at American plants. Following points are considered as most essential: (1) Improving quality to effect that high early strength cement will be manufactured more than ordinary standard portland cement; (2) Investigation of wet process and increase of fineness of raw mixture and cement; (3) Lowering of production costs by slurry filtration; (4) Increased application of closed circuit grinding and use of vibrator screens and classifiers at raw feed end and air separators for cement; (5) Improved dust collecting systems and greatest attention to safety devices.—A. E. BEITLICH

Researches on the rotary kiln in cement manufacture.—Part 13. GEOFFREY MARTIN. *Rock Products*, Feb. 28, 1931, V. 34, No. 5, p. 72.—In calculation of theoretical flame temperature in a rotary kiln, it is assumed that 1-lb. coal gives gaseous mixture CO_2, 2.624 lb.; N_2, 7.669 lb.; H_2O, 0.448 lb.; excess air, 0.537 lb.; total, 11.278 lb. Air, including 5 per cent excess is put at 10.478 lb. and it is assumed to have a temperature of 60° F. Highest temperature that can be attained by combustion gases is about 4,000° F. To calculate the maximum temperature, the mean specific heats between 4,000 and 60° F. of the components of the combustion gas are taken from the author's tables of specific heats. (The water is supposed to be gasified at 60° F. and then heated to the maximum temperature T.) Then approximate flame temperature of T deg. F. is calculated and found to equal 3763° F. In this calculation assumption is made that specific heats of gases follow a straight line law between 60° and 4000° F. and also that mean specific heat between 60° and 4000° F is same as that between 60° and 3763° F. These

assumptions introduce two slight errors. Formula for calculating flame temperature is modified to: Total heat equals heat liberated by 1-lb. of coal plus heat brought into furnace by 10.478-lb. of air, equals 12,600 B.t.u. plus QT. A table is appended showing the B.t.u.'s required for preheating 10.478-lb. of air from 60° F to various temperatures. By substituting in the formula various values of QT in table, approximate flame temperatures corresponding to different temperatures of preheating are found. More exact determinations are then made. The difference between the B.t.u.'s absorbed in heating furnace gases from 60° F to 1481° F., and 12,600 B.t.u.'s is heat absorbed in heating gas from 1481° deg. F to its final temperature of T deg. Final formula works out to: $T = \frac{7972 + Qt}{11.278 \times S_T} + 1481$ where T equals the flame temperature deg. F, Qt equals the B.t.u.'s brought into furnace by 10.478-lb. of air, and S_T mean specific heat of the combustion gases between 1481° and T deg. F. The various values of Qt taken from Table 1 and the varied values for S_T corresponding to approximate flame temperatures in the second table are substituted in the formula and different temperatures calculated. The final table shows flame temperatures beginning at 3769° F., when air is admitted at 60° F. to 5370° F. when the air is preheated to 2500° F.—EDMUND SHAW

High early strength cements and their increase in strength. P. MAY. *Tonind Ztg.* (Germany), Feb. 9, 1931, V. 55, No. 12, p. 171-3.—Strength measurements up to 5 years show a general increase. Sometimes temporary retrogressions are encountered.—F. O. ANDEREGG

The geology of gypsum. V. CHARRIN. *Rev. materiaux construction trav. publics* (France), March, 1931, No. 258, p. 99-101.—The deposits of gypsum which supply 2 million tons a year around Paris, in Lorraine and Provencales occur either in triasic or miocene formations.—F. O. ANDEREGG

The enumeration of French cement plants. PAUL RAZOUS. *Ciment* (France) Jan. 1931, V. 36, No. 1, p. 13-8; No. 2, p. 59-61.—A description of the properties and list of the different companies and plants making the following cements is given: natural, Roman, artificial (portland), high early strength, alumina, slag (using lime), grappiers (from flint nodules), mixed, white, iron (70 per cent portland clinker with 30 per cent granulated blast furnace slag), and blast furnace cement (contains less clinker than iron cement).—F. O. ANDEREGG

The new cement plant at Neuwied. C. E. PERES. *Tonind. Ztg.* (Germany), March 5, 1931, V. 55, No. 19, p. 275-7.—General plant description.—F. O. ANDEREGG

Testing protective coatings for concrete. P. MECKE. *Tonind. Ztg.* (Germany), March 16, 1931, V. 55, No. 22, p. 322.—To determine the solvent accurately in bituminous preparations 5 g. is placed in a small foil box containing a small roll of gauze. After drying at 100° C. for one hour with the cover removed, the bitumen is distributed over gauze. Then dry to constant weight at 110°. To test the resistance against chemical effects a mortar specimen is made 5x5 in. and a cylinder cemented to it with neat cement. After the cement hardens three coats are applied and the cylinder filled with the solution to be tested.—F. O. ANDEREGG

A new shaft kiln cement plant. HORST LAEGER. *Tonind. Ztg.* (Germany), March 16, 1931, V. 55, No. 22, p. 322-5.—Shaft kiln cement plant erected in Ankara (Angora) with rotating grate.—F. O. ANDEREGG

The different kinds of lime and their handling. F. EISEMANN. *Tonind. Ztg.* (Germany), March 19, 1931, V. 55, No. 23, p. 339-41.—A general description.—F. O. ANDEREGG

Cement synthesis in the four component parallelogram and the degree of lime saturation according to Kühl. M. SPINDEL. *Tonind. Ztg.* (Germany), March 12, 1931, V. 55, No. 21, p. 305-7.—Parallelogram show-

ing composition of cements in terms of lime, silica, alumina and iron oxide can also be used for Kühl's degree of lime saturation.—F. O. ANDEREGG

Industrial heating with ground coal in the mineral industries. J. DEFORGE. *Rev. materiaux construction trav. publics.* (France) Dec., 1930, No. 255, p. 469-73; Jan., 1931, No. 256, p. 5-10; Feb., 1931, No. 257, p. 49-53; March, 1931, No. 258, p. 101-5.—A careful analysis giving advantages and disadvantages.—F. O. ANDEREGG

Modern cement burning. HEINRICH LUFTSCHITZ. *Tonind. Ztg.* (Germany) Feb. 12, 1931, V. 55, No. 13, p. 184-6.—In Schröder shaft kiln dry raw material falls through a shaft kiln heated by coal dust falling through an annular space outside. Also the Rigby patent is used for spraying slurry into a kiln.—F. O. ANDEREGG

Determining the proportions of mortars and concretes. LUDWIG SCHUSTER. *Tonind. Ztg.* (Germany) Feb. 23, 26, 1931, V. 55, No. 16, 17, p. 224-5, 243-4. Examples are given where composition of old mortars and concretes are determined by making analyses for soluble silica, lime, etc. If possible, an analysis of aggregates simplifies solution of problem.—F. O. ANDEREGG

The new cement plant at Rodaun. K. A. GOSLICH. *Tonind. Ztg.* (Germany), March 16, 1931, V. 55, No. 22, p. 317-20.—Description of the erection of a new, dry process cement plant on site of the old one.—F. O. ANDEREGG

German specifications for alumina cement. HUGO VIERHELLER. *Tonind. Ztg.* (Germany), March 16, 1931, V. 55, No. 22, p. 325-6.—Fineness: not more than 2 per cent on the 75 and 15 per cent on 180 mesh sieve. Tests to be made at 15 to 20° C. Storage of specimens 24 hr. in damp closet and then under wet cloths. Minimum strengths are: at 24 hr., at 28 da. wet and at 28 da. combined storage: compression, 400, 500, 600 and tensile; 25, 30 and 40 kg. per sq. cm.—F. O. ANDEREGG

Rapid determination of magnesia in portland cement. CH. ROGOZINSKI. *Tonind. Ztg.* (Germany), March 23, 1931, V. 55, No. 24, p. 355-6.—After removing the silica, *N* NaOH is added till a precipitate forms and the supernatant liquor is clear. Filter into an excess 0.2 *N* HCl and titrate back with 0.2*N* NaOH and phenolphthalein—F. O. ANDEREGG

Measures for securing cement of uniform quality. W. SACH. *Tonind. Ztg.* (Germany), March 9, 1931, V. 55, No. 20, p. 291-3.—Proper feeding, weighing and mixing, obtained with suitable automatic apparatus are useful in securing uniform quality.—F. O. ANDEREGG

The uniformity of concrete and its control on the job. FRITZ EMPERGER. *Tonind. Ztg.* (Germany), March 30, 1931, V. 55, No. 26, p. 382-3—Cubes and beams are made and broken at seven days.—F. O. ANDEREGG

Alumina cement. M. KAUCHTSCHISCHWILI. *Siemens-Z.* 1930, No. 9, 10; *Tonind. Ztg.* (Germany), Dec. 18, 1930, V. 54, No. 101, p. 1565-7; No. 102, p. 1585-6.—Description of general properties and of manufacture.—F. O. ANDEREGG

Studies on high-iron oxide cements—Part 1. SHOICHIRO NAGAI AND KEI-ICHI AKIYAMA. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), Feb., 1930, V. 33, Supplemental Binding No. 2, p. 47-9B.—Two high-iron oxide cements, one of the iron cement type and one of Kuhl cement type were prepared of two pozzuolanas, analyses of which are given and limestone. Composition and properties of burned clinkers were studied and compared with commercial products. Normal and small-piece testing method were used for determination of compressive strength. (cf. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), 1930, V. 33, p. 286-90, 421-8; JOURNAL A. C. I., Feb., 1931, V. 2, No. 6, *Abstracts* Sect., p. 162.)—A. E. BEITLICH

Cement-mill expansion program reflects vision of designing engineers. W. E. TRAUFFER. *Pit and Quarry,* March 11, 1931, V. 21, No. 12, p. 67-78.—Improvements in plant of Keystone Portland Cement Co., at Bath, Pa., include enlargement of stone storage, a second overhead traveling crane,

two raw-grinding and two finish-grinding mills, additional slurry silos, two kilns and stacks, enlargement of the coal-pulverizing equipment, additional cement-storage silos and packhouses, and enlargements to crusher, mill and kiln buildings. Raw mills are opposite stone storage, coal mill is opposite coal storage, discharge ends of kilns and finish mills are opposite clinker storage, and slurry silos are between raw mills and feed ends of kilns. All raw material comes from one quarry and as sent to mill is practically correct composition for high grade cement.—A. J. HOSKIN

Microscopical, optical and X-ray investigations concerning the question: "What is alit?" I. WEYER. *Zement* (Germany), Jan. 29, 1931, V. 20, No. 5, p. 96-100.—Alit, separated from commercial clinker, was found to contain considerable amounts of alumina. This fact led to beliefs that tricalcium aluminate can enter into combination with tricalcium silicate. In experiments made to study problem, various mixtures of pure tricalcium silicate and tricalcium aluminate were mixed and burned several times at different temperatures and products investigated by X-ray analysis. None of mixtures showed any characteristic lines besides lines for tricalcium aluminate and tricalcium silicate. In mixtures with only 6.89 per cent tricalcium aluminate (16:1), it was impossible to detect this compound. Microscopic studies showed that a recrystallization of tricalcium silicate takes place in such melts which fact does not change its optical characteristics.—A. E. BEITLICH

Petrographic investigation of fused high alumina cements. KATSUZO KOYANAGI. *Kogyo Kwagaku Zasshi, J. of Soc. Chem. Ind.* (Japan), Sept., 1930, V. 33, Supplemental Binding, No. 9, p. 352-8B.—Investigations and experiments by Gruen, Dyckerhoff and Biehl in studies of chemical constitution of high alumina cements are described. Efforts were made by author to prepare synthetically a melt of composition: 47 per cent CaO, 44 per cent Al_2O_3 and 9 per cent SiO_2, which is similar to French Ciment fondu. After different heat treatments thin sections were studied under microscope and optical properties were determined. Furthermore melts of following compositions were prepared: $3CaO.5Al_2O_3$, $CaO.Al_2O_3$, $3CaO.Al_2O_3$ and $5CaO.3Al_2O_3$ and microscopically investigated.—A. E. BEITLICH

Studies of the method by Lawrence Smith for the determination of alkalies. CURT PRUSSING. *Zement* (Germany), April 16, 1931, V. 20, No. 16, p. 360.—The method for determination of alkalies which was proposed for German standard specifications for chemical analysis of portland cement (Burchartz, *Zement* (Germany), March 19, 26, 1931, V. 20, No. 12-13, p. 258-60, 290-2), was checked and it was found that temperature for ignition of alkalies is too high. Great amounts of alkalies are volatilized. Mixtures of sodium chloride, potassium chloride and ammonium chloride were ignited at 932 to 1922° F. and loss on ignition was determined. Author proposed heating of sample in electric crucible furnace provided with a rheostat to regulate temperature between 700 and 750° F.—A. E. BEITLICH

A mixed cement of low solubility of portland cement and arsenic trioxide. DONOVAN WERNER AND STIG GIERTZ-HEDSTROM. *Zement* (Germany), April 23, 1931, V. 20, No. 17, p. 384-8.—Investigation of possibilities of use of a mixture of portland cement and arsenic trioxide yielded following: Mixed cement with 30 per cent As_2O_3 is quick setting (5 minutes); it can be made normal setting by heating in closed container. Compressive and tensile strengths are lowered by additions from 10 to 15 per cent As_2O_3; with increasing additions strength increases too and reaches maximum with 30 per cent. Above 30 per cent strength decreases again. Strength is always slightly lower than strength of pure cement. Solubility of lime of portland cement is greatly reduced by additions of As_2O_3. Best composition is 70 per cent portland cement and 30 per cent arsenic trioxide which produces, after heating, a very slightly soluble normal setting cement with strength properties above limits of specifications.—A. E. BEITLICH

The determination of the uncombined lime. Supplemental discussion of the glycerol-tartaric acid method of H. Rathke. K. SCHINDLER. *Zement* (Germany), April 23, 1931, V. 20, No. 17, p. 389-90.—A new procedure is described which overcomes difficulties and inaccuracies of method by Rathke who modifies Emley's method by using tartaric acid instead of ammonium acetate for titration of uncombined lime. Greatest source for errors is tendency of calcium glycerate to crystallize before all calcium oxide is dissolved thus preventing complete solution. It affects especially standardization of tartaric acid. Procedure is same for cement and similar materials.—A. E. BEITLICH

Investigation of portland cement and its constituents by means of measurements of vapor pressure. Part 2. F. KRAUSS AND G. JOERNS. *Zement* (Germany), April 2, 9, 1931, V. 20, No. 14, 15, p. 314-7, 341-3.—Preliminary investigation and experiments were reported in previous article (*Zement* (Germany), Nov. 6, 1930, V. 19, No. 45, p. 1054-5; JOURNAL A. C. I., Feb. 1931, V. 2, No. 6, *Abstracts*, p. 164). Work was continued and vapor pressure curves determined for gypsum, calcium carbonate, calcium hydroxide and hydrates of silica, alumina and ferric oxide. Synthetic melts of tricalcium silicate, tricalcium aluminate and compound $8CaO.2SiO_2.Al_2O_3$ were prepared from pure raw materials and hydrated. Products of hydration were decomposed and characteristics of vapor pressure curves were studied. Similar tests were conducted with portland cement. Work will be continued.—A. E. BEITLICH

Contribution to the hardening problem of portland cement. HANS KUHL. *Zement* (Germany), March 12, 1931, V. 20, No. 11, p. 234-9.—Contrary to Tippmann's theory, (cf. *Zement* (Germany), Dec. 25, 1930, V. 19, No. 52, p. 1225-34) chief products of hydration of portland cement are calcium hydrosilicates, not calcium hydroxide and silica gels. Cements of normal consistency do not have a high viscosity. Special consideration is given to description of 2 different modifications of calcium hydroxide. Existence of amorphous calcium hydroxide with a specific gravity of 2.08 is denied. Experiments for preparation of this form were made according to Tippmann's directions but with negative results. Cement was mixed with water and with weak and saturated gypsum solutions. Products of hydration were examined under microscope at frequent intervals. No distinct difference could be detected. Author obtained small crystals of $Ca(OH)_2$ from calcium oxide and water which formation was claimed to be impossible by Tippmann. Statement that 2 modifications of calcium hydroxide are chief constituents in hardening process is attacked. This property is attributed to calcium hydrosilicates. The microscopic method used by Tippmann is criticized. Theoretical deductions and experimental findings do not harmonize.—A. E. BEITLICH

The effect of different kinds of gypsum on setting of cements. KATSUZO KOYANAGI. *Zement* (Germany), March 5, 1931, V. 20, No. 10, p. 215-7.—In experiments with a portland cement clinker and gypsum, hemihydrate, dehydrated anhydrite and natural anhydrite, retarders were either ground together with clinker or fine ground materials were mixed together in a laboratory size mixer. Setting properties of mixtures with 0 to 10 per cent SO_3 were determined. A certain percentage was found where retarding effect is greatest. This is 5 per cent SO_3 for natural anhydrite, 2 per cent for gypsum, 1.5 per cent for hemihydrate and 1.2 per cent for dehydrated anhydrite. Effect of retarders was greater when mixed with fine clinker than in case where both were ground together. Experiments A. Heiser (cf. *Zement* (Germany), July 1930, V. 19, No. 29, p. 679-80; JOURNAL A. C. I., Oct. 1930, V. 2, No. 2, *Abstracts*, p. 34), yielded entirely different results. His tests were made on large scale under different temperature conditions and his opinion is supported by Grimm, Forsen and Tippmann.—A. E. BEITLICH

Effect of storage on properties of standard cements. RICHARD GRUEN AND H. MANECKE. *Zentralblatt der Bauverwaltung* (Germany), 1930, V. 50, p. 485, reviewed in *Zement* (Germany), Jan. 22, 1931, V. 20, No. 4, p. 86.

—Storage of cements at 18° F. resulted in decrease of about 8 per cent in 7-day tensile strength and increase of about 20 per cent in 3-day compressive strength. Storage in closed room had no harmful effect on strength properties. Careless storage in single sacks in open shed led to decrease of tensile and compressive strength, especially when lumps were not removed by sieving. Greatest losses after 3 months storage were 17 per cent for tensile strength and 14 per cent for compressive strength.—A. E. BEITLICH

Loss in strength of cement through storage. WALTER OBST. *Betonwerk*, March 29, 1931, V. 19, No. 13, p. 173.—Cement that does not deteriorate in storage is unknown, but fresh cement is not always constant in volume and a period of storage is helpful. Storage in warehouse in a thick layer and periodic turning over by plowing does much to prevent "efflorescence." This method also increases strength. For best practice cement should not be stored more than three months and never longer than nine months. Twelve months decreases strength up to 30 per cent. Storage rooms should be free of draft, cement should be placed on planking raised up from floor and bags of cement should be placed free of wall and piled not higher than 10 high. Paper bags are better than cloth bags for keeping cement in good condition.—H. FRAUENFELDER

Changes of setting time of portland cement. HELLMUT WEITHASE. *Zement* (Germany), Feb. 26, 1931, V. 20, No. 9, p. 187-92.—Thirty references are given to papers dealing with changes in setting time of portland cement during storage. Phenomena are explained in many different ways. To clear this question comprehensive tests with 55 cements were undertaken. Cement was spread in thin layers of about 1 or 2 in. and setting times were determined within very short intervals of storage. Certain very fine cements show earlier initial set in air storage but retarded final set. Changes seem to be caused in the first place by chemical composition, especially lime content of cement and also by effect of carbon dioxide of air on cement.—A. E. BEITLICH

The Rigby-spray method in cement industry. S. I. M. AULD. *Zement* (Germany), Feb. 12, 1931, V. 20, No. 7, p. 142-4.—Greater heat economy and increased efficiency is obtained by spraying slurry into kilns by means of compressed air. Great surface of dispersed fine materials facilitates drying. Kiln gas temperature is lowered causing less coal consumption and smaller radiation losses. Calcination zone of kiln is as long as in ordinary kiln. Only 26 per cent of total length is used for drying. Nozzles under pressure of 5 atmospheres can be easily replaced without interruption of operation and at low costs. Installation of fan is advisable to compensate for decreased draft on account of lower temperature of kiln gases.—A. E. BEITLICH

The last word in the alit problem. A. GUTTMANN AND F. GILLE. *Zement* (Germany), Feb. 12, 1931, V. 20, No. 7, p. 144-7.—After studying latest research results of American and German investigators on subject of existence of tricalcium silicate and presenting data on X-ray diffraction patterns in several tables, authors draw following conclusions. (1) No doubt exists about existence of tricalcium silicate. (2) Alit, chief constituent of high lime portland cements, is neither a dicalcium silicate which is rich in free lime (Dyckerhoff), nor a solid, solution of a high lime aluminate, (Jaeneckeit), and tricalcium silicate (Kuehl), nor a solid solution of chiefly tricalcium silicate and tricalcium aluminate (Guttmann and Gille), nor a solid solution of Jaeneckeit and dicalcium silicate (Jaenecke), nor is dicalcium silicate chief constituent of alit (Nacken), but alit is tricalcium silicate. (3) Part of alumina is combined as celit (chiefly $4CaO.Al_2O_3.(Fe,Mn)_2O_3$). Remaining part is present as tricalcium aluminate and partly as $5CaO.Al_2O_3$.—A. E. BEITLICH

Fine cement. A. B. HELBIG. *Zement* (Germany), Jan. 22, 1931, V. 20, No. 4, p. 75-8.—Contrary to findings of Hans Kühl and A. Hauenschild, cement strength increases with increasing fineness, especially in early ages. Decrease in strength which was observed by above investigators in cases of extremely fine cements, is due to a partial hydration of fine particles by moisture of air in air separator. Commercial cements were separated into several fine fractions

by means of a large scale air separator. Tables show fineness of separation, water for normal consistency and compressive and tensile strength data up to 28 days in water and combination storage. Fineness affects water permeability. Case is cited where fine grinding increased strength in ratio 1:1.9 while water permeability was decreased in ratio 56:1. Diagram shows relation between particle size of cement and best suitable diameter of steel balls in mill. —A. E. BEITLICH

Fine cement. HANS KUHL. *Zement* (Germany), Feb. 19, 1931, V. 20, No. 8, p. 169-70.—In answer to article by A. B. Helbig in *Zement* (Germany), Jan. 22, 1931, V. 20, No. 4, p. 75-8, author states that Helbig did not work with cements of such extreme fineness and therefore did not notice a decrease in strength in very fine range. Inner surfaces of cements, used by Helbig, were calculated by means of formula derived by Kuhl and Tokune. Finest cement had surface of 216.5 sq. in. A drop in strength was observed only in cases of cements with inner surfaces of 500 sq. in. and higher. Cements investigated by author were not affected by moisture; air for separation was absolutely dry.—A. E. BEITLICH

New Mexican cement plant serves west coast territory. *Concrete*, March, 1931. V. 38, No. 3, p. 77.—Wet process cement plant is nearing completion at Hermosillo, Sonora, Mexico. Present capacity is 500 barrels per day (one 8 by 150-ft. kiln), provisions being made for increasing capacity at any time. Raw materials are stone and clay. There are two gyratory crushers (one 12 in. and one 4½ in.), and two 6 by 22-ft. compartment mills (one for raw and one for finishing grinding). Power is furnished by three 360 h. p. Diesel engines direct-connected to Westinghouse generators.—V. E. BUTLER

Technical progress reports. Volume 25. Cement. "Technische Fortschritts Berichte. Band 25. Zement." F. WECKE, 1930, Th. Steinkopff, Dresden and Leipzig, (Germany), RM. 6.00. Reviewed in *Zement* (Germany), Feb. 12, 1931, V. 20, No. 7, p. 156.—Valuable data are presented on portland cement, raw materials, manufacture and testing, and high early strength cements, special cements and blast furnace slag cements are discussed. Numerous literature references, complete up to 1929 aid study of progress and problems in cement manufacture and cement chemistry. Statistical data are furnished on import and export trade since 1900.—A. E. BEITLICH

The "Delta" rocker chute. H. WALTER. *Zement* (Germany), Feb. 26, 1931, V. 20, No. 9, p. 201-2.—New improved rocker chute is especially suitable for transportation of large amounts of dusty, pulverized and broken materials, Design and construction make operation very economical. Machine is applicable for conveying of hot clinker in cement industry.—A. E. BEITLICH

The degree of lime saturation. HANS KUHL. *Zement* (Germany), Feb. 5, 1931, V. 20, No. 6, p. 123-5.—In criticism of proposed changes of "degree of lime saturation" by Hess (cf. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 28-32; JOURNAL A. C. I., April, 1931, V. 2, No. 8, *Abstracts*, p. 214), author agrees with proposal to express degree of lime saturation by a number 100 times as big as previously used but he criticizes attempts of Hess to make changes in derived equation, especially to give numbers with too many decimal places. Latter procedure would give a false impression of accuracy of calculation method.—A. E. BEITLICH

New cement plant in Dumesnil (Argentina). *Zement* (Germany), Feb. 5, 1931, V. 20, No. 6, p. 134.—At modern wet process portland cement plant recently completed in close vicinity of limestone quarries near Dumesnil, aerial cable ways deliver limestone and clay in cars. Clay slurry is ground together with limestone. Material passes through kiln bins into 9 rotary kilns, 223 ft. long and 9.8 and 11.8 ft. in diameter. Each kiln produces 220 tons of clinker in 24 hours. Fuel consumption is 44 tons of crude oil. Finished product is pumped to cement storage bins. Plant is equipped with very modern cement laboratories, machine shops, water tower, cooling tower and modern power house.—A. E. BEITLICH

Finding undeveloped sources of fuel for cement plants. E. T. ELLIS. *Concrete*, Feb., 1931, V. 38, No. 2, p. 91.—Certain so-called low grade fuels are apt to be overlooked by cement manufacturers, but can, if manipulated in certain easy ways, be utilized advantageously for burning, either alone or combined with coal and other materials usually employed. The compositions and names of these many materials vary widely in different parts of the world, but analyses in many instances are available, and by examining these together with materials themselves it is possible to judge whether any offer of such materials should be declined or accepted. Materials discussed are: Anthracite culm, anthracite slush or silt, "batts" from coal cleanng dumps, colliery slurry, colliery smudge, refuse from tipple picking tables, steam duff, tip breeze, and washery waste.—V. E. BUTLER

Discussion of the new proposed German standard specifications for reinforced concrete. OTTO GASSNER. *Zement* (Germany), Feb. 26, 1931, V. 20, No. 9, p. 198-200.—Changes and additions to tentative specifications of the German Committee for Reinforced Concrete, Part A, Specificatons for Construction of Structures of Reinforced Concrete are criticized. Definition of standard cement is slightly changed and high early strength cements are included. Natural cements are not mentioned. Exact control of gradation of aggreates, especially for important structures, wet concrete and under-water cast concrete are demanded. Use of broken volcaneous materials and pumice are admitted as aggregates. More consideration is given to impurities of aggregates. No boiler cinders are allowed for reinforced concrete. In structures exposed to high temperatures reinforced concrete must contain aggregates with small expansion coefficient and low heat conductivity. Quartzy aggregates should not be used in chimney construction. Insufficient definition of degree of impurity of mixing water is criticized.—A. E. BEITLICH

Only active cement plant in Arkansas. J. F. KAUFMAN. *Rock Products*, March 28, 1931, V. 34, No. 7, p. 41.—Distinguishing feature of operation of Arkans Portland Cement Co.'s mill at Okay, Ark. is filtering of slurry in three 12-disk filters with 7200 sq. ft. filtering surface. Without filtration the slurry, made from chalk and clay, had a moisture content of 48-50 per cent and was hard to handle because of viscous nature. Filtration reduced moisture to 28½-29 per cent. Filter cake is fed to kiln by a Bessemer feeder. Capacity of filter depends on temperature of slurry. Heat of waste gases brings the water to 150° F. Cast iron lifters in the kiln keep the slurry moving forward and prevent the formation of mud rings.—EDMUND SHAW

Contribution to the question of kilns for the manufacture of fused cements. E. SCHIRM. *Zement* (Germany), Feb. 5, 1931, V. 20, No. 6, p. 118-23.—Various kinds of high alumina cement kilns have their advantages and disadvantages (1) In shaft kiln with water jacket, raw mixture consisting mostly of limestone and bauxite becomes pasty which fact causes considerable difficulty. (2) Improved shaft kiln has separate horizontal fusion chamber. Material in vertical part is only preheated and not fused. (3) Still better is kiln so constructed that pasty zone is prevented. (4) Another shaft kiln consists of 3 different chambers. Material is heated and partly decomposed in top chamber, then fed to second section on a water cooled grate. (5) Greatest difficulty with rotary kiln is formation of back rings which may block entire kiln. (6) An improvement was developed in France by installing a longer kiln head with hottest part of flame where material leaves kiln. (7) Better heat economy is obtained by installing a recuperator. (8) Kiln has been designed in which bauxite and limestone are heated separately in 2 concentric tubes. Heated materials are emptied into hearth furnace where mixture is burned completely. —A. E. BEITLICH

Chemical reactions in the setting of cement. LINCOLN T. WORK AND FRANKLIN P. LASSETER. *Concrete*, March, 1931, V. 38, No. 3, p. 81-86, No. 4, p. 89-92, No. 5, p. 79-84.—Investigation of chemical reactions that take place during setting and hardening of portland cement is abstracted by the authors.

In studying state of combination of water, following methods were tested: 1. Extraction with absolute alcohol. 2. Distillation using benzol, toluol and xylol. 3. Vacuum drying. 4. Oven drying in dry air at 90, 105 and 130 deg. C. The test of oven drying at 105° C. was chosen as standard. Total water was determined at 850° C. Under those conditions a small amount of moisture will remain, but calcium sulfate should not be decomposed appreciably. Therefore, difference representing combined water should be essentially correct. Ammonium acetate method for lime in clinker has been adapted to measure hydrated lime in a set cement. Samples with varying water-cement ratios using each of the compounds tricalcium aluminate, tricalcium silicate, and β dicalcium silicate, and four commercial cements, were studied to ascertain nature of chemical reactions and rate of hydration and hydrolysis during setting and hardening. Tricalcium aluminate hydrates to $3CaO.Al_2O_3.6H_2O$ and this on calcination yields lime and an apparent residue of $5CaO.3Al_2O_3$. This hexahydrate can be partially dehydrated in vacuum at 240-250° C. With water-cement ratios of 0.5 and greater, tricalcium aluminate was practically completely hydrated to hexahydrate in 28 days. Tricalcium silicate reacts both by hydration and hydrolysis. During early stages of hydrolysis approximately one molecule of water combines with silicate for each molecule of lime liberated. Beta dicalcum silicate was shown to react chiefly by addition of water and released very little lime for several months. Pure compounds and cements yielded a typical curve for percentage of combined water against water-cement ratio. Starting at the origin, values of combined water rose with water-cement ratio up to a ratio of 0.2 to 0.4, after which they became practically horizontal. In case of the aluminate, reaction at low water-cement ratio consumed all of the water, but that was not found to be the case with other pure compounds or with cements. Hydration was rapid with aluminate, slower with tricalcium silicate and very much slower with dicalcium silicate. Tricalcium aluminate yielded no lime in setting. Tricalcium silicate yielded lime more or less similar in amount to combined water value and beta dicalcium silicate yielded but small quantities of lime up to three months. Combined water values for cements were similar to those for tricalcium silicate but rate of hydration varied directly with fineness. Lime values were somewhat similar, save for a dip in the curve at low water-cement ratios which appears to be due to selective hydration of aluminate. Comparison of hydration and hydrolysis of cements was made with strength of a standard concrete using same cements at water-cement ratio of 0.55. Fine cements reacted more completely in early stages and developed high strength. Coarse cement reacted slowly at first but for a longer time. Later reaction did not develop an equivalent strength.—V. E. BUTLER

Constitution of portland cement studied by X-ray method. L. T. BROWNMILLER AND R. H. BOGUE. *Concrete*, Feb. 1931, V. 38, No. 2, p. 85, March, 1931, V. 38, No. 3, p. 89.—Objectives in research were (1) to obtain additional information on theories established by other investigators, and (2) examination of a large number of commercial clinkers by x-ray and chemical methods to find out exactly what compounds could be positively identified in each and nature of agreement between such x-ray analysis and constitution as computed from chemical analysis. It was necessary to determine efficiency of x-ray method to decide pattern of each compound in presence of other compounds of cement. An examination of composition $3CaO + SiO_2$ proved that $3CaO.SiO_2$ exists and can be formed by heating together, under proper conditions, of a properly proportioned mixture of lime and silica. An examination of the system $3CaO.SiO_2$—$2CaO.SiO_2$—$3CaO.Al_2O_3$ shows that solid solution does not occur between either of above silicates and aluminate in large amounts. Mixtures at equilibrium contain only those three compounds. In commercial cement clinkers free CaO was not found by x-ray method. Study of identification of that compound indicates that a percentage of CaO of 2.5 could be recognized. Therefore, it is demonstrated that free CaO is not commonly present in commercial clinkers in amounts as high as 2.5 per cent. The x-ray

diffraction patterns of 28 commercial portland cement clinkers, representing many types of raw material and processes of manufacture—both domestic and foreign—were obtained. Results obtained by x-ray methods define major constitution of portland cement clinker and indicate most abundant constituents are tricalcium silicate and beta dicalcium silicate; that there are normally present, in addition, tricalcium aluminate, tetracalcium alumino ferrite and magnesia; and that free CaO is not normally present in amounts as great as 2.5 per cent.—V. E. BUTLER

Free lime in portland cement clinker and soundness of cement. KATSUZO KOYANAGI. *Rock Products*, March 28, 1931. V. 34. No. 7, p. 74.—There are two opinions of free lime in cement clinker. Rohland thinks unsoundness comes from lime that is neither dissolved in other constituents nor chemically combined, and Kuhl thinks that lime in cement is never free and that unsoundness comes from "explosive grains" containing excess lime in solution or combination. Japanese cement burners test clinker by touching it to the tongue. If saliva is absorbed quickly clinker is thought unsound. To test this, Emley's method of determining free lime, as modified by Rathke has been used by the author in tests of sound and unsound clinkers which included determinations of porosity and free lime. Porosity was determined by immersing in water-free turpentine oil for 24 hr. and then boiling ½-hr. and cooling and weighing. Soundness was determined by water test, boiling and a modification of LeChatelier's test. Further research showed clinkers could have low porosity and high free lime content. These were hard burned clinkers with high iron oxide content except one with extraordinarily low lime content and overburned. In mixtures of sound with unsound cement, with free lime running from 0.45 to 0.81 per cent, samples were sound; from 0.99 to 1.88 per cent, samples were unsound, increasingly so as free lime increased. A third series of tests showed effect of storing unsound cement on paper in laboratory so that free lime had a chance to carbonate. The temp. was 73-86° F. and the moisture, 80-100 per cent normal. In 3 da. free lime fell from 2.44 per cent, to 1.49 per cent and in 40 da. to .30 per cent. All samples taken at 20 da. and after were sound. At a fineness of 4 per cent on a 4900-mesh (metric) sieve cement becomes unsound in boiling test when free lime exceeds 1 per cent and unsound to both boiling and water tests when it exceeds 2.2 per cent. Whether this free lime is absolutely free or is combined in explosive grains is not known exactly, but it is certain that it affects soundness of cement very much.—EDMUND SHAW

A tentative modification of the free lime method. H. R. BRANDENBURG. *Rock Products*, March 14, 1931, V. 34, No. 6, p. 68.—Use of barium chloride as a reagent to increase solubility of CaO in glycerol-alcohol mixture in determining free lime is suggested. The $BaCl_2$ is made anhydrous by holding at 140-150° C. for several hours. Then it is cooled, ground and stored in a desiccator. Tests were agitated on the base of a Ro-tap screening machine, to which had been clamped a sheet metal holder that would take six flasks. Standards were run on freshly calcined CaO, from .05 to 25 g., with and without $BaCl_2$. Solution of CaO was extremely rapid, permitting additions of acetate solution to be made at 2 to 5 min. intervals, care being taken not to run in an excess. CaO values were found to be a trifle higher than with standard method. On materials requiring from 2.0 to 2.5 cc. of the acetate solution, time with $BaCl_2$ varied from 25 to 40 min. as against 2½ to 4 hr. by the standard method. Method has been used successfully on clinkers, hydrated limes and plasters.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part 14. Flame temperatures obtained in practice in the cement rotary kiln. GEOFFREY MARTIN. *Rock Products*, March 14, 1931, V. 34, No. 6, p. 41.—Actual temperatures observed by the B. P. C. R. A. by optical pyrometer were 2759, 2692, 2536 and 2835° F. Prof. Nacken found a temperature of 2606° F. and E. C. Soper 2587° F. with the same instrument. Prof. Nacken's figures are taken as the mean flame temperature in ordinary kiln. It is assumed that

11.278 lb. combustion gas is produced by burning 1 lb. coal in t sec. In insulated enclosure, with 10.478 lb. air preheated to 400° F. gas will rise 3977° F. In the actual kiln so much heat is lost by radiation, conduction and convection that temperature is reduced to 2600°. Heat is lost by (1) external radiation, (2) radiating from hot flame to cooler parts of kiln and (3) communicating by conduction and convection to material in clinkering zone. (1) is waste but (2) and (3) can be usefully employed under limitations. At clinkering temperature clinker tends to evolve heat. In practice amount of clinker turned out of flame sinking through different temperatures can be calculated.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part 15. Weight of clinker producible using combustion gases of different flame temperatures and supplying 10.478 lb. air per 1 lb. of standard coal burned. GEOFFREY MARTIN. *Rock Products*, March 28, 1931, V. 34, No. 7, p. 68.—Main heating agent in kiln is hot gases into which powdered coal oxidizes. Each pound coal yields 11.278 lb. gas which, in the ordinary cement kiln, starts at 2600° F. and imparts its heat to material as it travels. The kiln may be divided into two parts, A where the temp. is above 1481° F. and B where it is below this. Limestone and chalk will decompose only in A. Heat in B is useful in drying and heating materials preparatory to clinkering in B. Quantity of clinker formed is measured by the amount of heat absorbed by the material in A between 1481° F. and the clinkering temp., 2498° F. This quantity of heat, Q equals $11.278 \times S \times (T - 1481°) \times$ B.t.u., where T is the max. temp. and S is specific heat. To form 1 lb. clinker requires 918.6 B.t.u., so weight of clinker W, formed by burning 1 lb. standard coal to its equivalent of 11.278 lb. gas equals Q, divided by 918.6. If coal is burned in t sec., amount of clinker produced in 1 sec. is $\frac{W}{t}$. As ideal kiln would require only 6.36 tons coal, ordinary kiln making 100 tons clinker for 28 tons standard coal is only 22.7% efficient. Computed results show that heating in kiln is by hot gas and that clinker output is measured by heat loss of gas as it travels down kiln. —EDMUND SHAW

The upper SO_3 limit in portland cement. F. O. ANDEREGG. *Rock Products*, Feb. 28, 1931, V. 34, No. 5, p. 76.—Better retardation of finely ground high-early-strength cement is reason for adding more gypsum to clinker than has been permitted in past. Question arises as to safe upper limit of SO_3 since excess gypsum is known to reduce durability of cement, probably by crystal growth. Tests were made with cement flour, air-separated so practically all particles were below 10 microns in diameter. Oversize was reground and separated to give a cement containing a fraction less SO_3. When cement is gaged with water a low sulphate form may be present at first but it changes over to more stable form which may grow into crystals large enough to be identified under microscope. In view of the test results the proposed A. S. T. M. specifications limiting SO_3 to 2½ per cent in early-strength cements appears conservative, but greater amounts are undesirable. Alumina content of clinker should be kept within reasonable limits. From a study of the rate of hardening of specimens made from cement flour it is thought that indurating reactions of silica gel play important role in hardening of this cement.—EDMUND SHAW

MISCELLANEOUS

Separation of calcium and magnesium by molybdate method. R. C. WILEY. *Ind. Eng. Chem.*, Analytical Edition, April, 1931, V. 3, No. 2, p. 127-9.—Calcium molybdate is quite insoluble in a solution that is nearly neutral. Calcium may be determined accurately by precipitating as molybdate. Presence of residual ammonium molybdate in washings from calcium molybdate does not prevent precipitation of magnesium as magnesium ammonium phosphate, and presence of ammonium salts does not interfere with determination of calcium as molybdate. Calcium and magnesium may be

determined in much less time when molybdate method for calcium is used than when calcium is precipitated by the oxalate method.—ROY N. YOUNG

Protective coatings. KARL MEYER. *Zement* (Germany), Jan. 29, 1931, V. 20, No. 5, p. 109-10.—It is claimed that protective media such as paraffine and asphalt, proposed by Goslich (cf. *Zement* (Germany), June 1930, V. 19, No. 24, p. 567-8; *Journal A. C. I.*, Oct. 1930, V. 2, No. 2, *Abstr.*, p. 38), do not give satisfactory service for protection of concrete and reinforced concrete. Better results can be obtained with modern cold asphalt paints. Goslich claims that coatings consisting of hydrocarbons (paraffines) are highly resistant in aggressive waters and moist air. Several examples are cited where such media have been used with great success.—A. E. BEITLICH

Pulverized coal fuel in the mineral industries. J. DEFORGE. *Rc. materiaux construction trav. publics*, (France), December 1930 to March 1931.—Conclusions drawn upon discussing in detail economy of pulverised coal as a fuel are: Advantages (a) more complete evolution of heat corresponding to calorific value of the fuel; (b) possibility of utilizing inferior grade fuels, not acceptable for use otherwise. (c) simplification of firing; instantaneous ignition and extinction; a specially valuable ease of operation under certain conditions of installation. Disadvantages (a) accident risk; (b) flue dust difficulties; (c) the high cost of installation. Accidents can be greatly reduced in using adequate precaution. Special devices cope with flue dust problem. High initial cost is frequently more than compensated through economies in using a cheaper fuel or through lower fuel consumption. Though in every case, a study should be made of special conditions pertaining to installation in question, French industries should take advantage of this most rational method of utilizing coal.—M. A. CORBIN

Mechanism of combustion of individual particles of solid fuels. DAVIS F. SMITH AND AUSTIN GUDMUNDSEN. *Ind. Eng. Chem.*, March, 1931, V. 23, No. 3, p. 277-285.—Descriptions in detail of unique apparatus and methods employed in a fundamental study of combustion of solid fuel are included. Effects of varying particle size, air velocity, temperature and humidity on rate of combustion of solid fuel under carefully controlled conditions have been investigated and considerable light is thrown upon the mechanism of these important reactions. Briefly the procedure consists of heating a weighed sphere of fuel to the desired temperature in inert gas, suddenly blasting the particle with an air stream of given velocity and temperature for a very short interval of time, quickly quenching the particle in inert gas, again weighing to determine amount of reaction, and carefully obtaining records of particle surface temperature during combustion. Most of work was done with nearly pure carbon shaped from electrode carbons such as used in arc lamps for projection. In addition, however, fuels of known ash content were investigated. Following conclusions were reached: (1) The specific surface-reaction rate for carbon spheres burned in dry and moist air is a complex function of particle size. It is enormously larger for a small particle than for a large one. (2) The temperature of the surface increases as the particle is reduced in size. (3) At lower velocities, below about 4 ft. per sec. as measured at 24 deg. C., reaction rate increases rapidly with increase in velocity, independent of other variables. This decreases continuously as the velocity increases until in high-velocity range a large increase in velocity increases reaction rate only slightly. (4) Size of particle, independent of other variables, has an effect on specific surface-reaction rate; that is, for the same air velocity and same surface temperature, a small particle has a higher specific surface-reaction rate than does a larger one. (5) Carbon spheres of surface area 70 to 10 sq. m.m. burn faster in dry air than in moist air. (6) The surface temperature is higher when the particles are burned in moist air. (7) An increase in surface temperature, for particles burned in dry air, increases the reaction rage—that is, the temperature coefficient is positive. Maximum temperature coefficient is associated with small particle size, high air velocity, and high surface temperature. (8) An

increase in surface temperature for particles burned in wet air at low velocities decreases the reaction rate, but for higher velocities the reaction rate increases with rise in surface temperature. (9) A mechanism explaining the observed effects of water vapor on the combustion of electrode-carbon spheres burned in wet air is suggested.—ROY N. YOUNG

Accurate air separator for fine powders. PAUL S. ROLLER. *Ind. Eng. Chem.*, Analytical Edition, April, 1931, V. 3, No. 2, p. 212-16.—Apparatus separates quantitatively a 1-kg. charge of fine powder into a series of fractions beginning 0-2.5 microns in size. Except for finest fractions, at or below 5 microns, where attrition by air current takes place in case of soft powders, particle sizes separated are very homogeneous within limits given by Stokes' law. Several causes affect rate of separation, but most important is that of rate of air flow. Under similar conditions rate of separation is proportional to air flow. Depending on the latter, initial rates of separation of particle-size fractions have been made up to 135 grams per hour at a flow of 140 liters per minute. Continuous separation can be effected by use of an offset separator tube with separate collection of oversize. At a 30-micron particle size cut of a portland cement powder, with an air flow of 500 liters per minute, equivalent rate of feed was 5.4 kg. per hour, while rate of recovery was 0.92 kg. per hour. —ROY N. YOUNG

Effect of mica and iron-mica on strength of mortar and concrete. G. KATHREIN. *Zement* (Germany), Feb. 19, 1931, V. 20, No. 8, p. 174-7.—Experiments at Institute for Mineralogy and Testing Materials at Technische Hochschule in Vienna (Austria) revealed interesting facts concerning effect of mica on concrete properties. Sand in 1:3 mortar specimens was replaced in certain proportions by mica. Compressive strength is lowered by 1 per cent mica; greater percentages cause rapid decreases in compressive strength. Tensile strength and weight per volume are increased by 2 to 5 per cent mica, decrease in strength takes place above 10 per cent. Similar tests were conducted with concrete specimens. Compressive strength is lowered with increasing W/C ratio and decreasing weight per volume. In general, mica has harmful effect on compressive strength and sometimes slightly improving effect on tensile strength. Experiments with iron-mica led to similar results.—A. E. BEITLICH

The rate of calcination of limestone. G. G. FURNAS. *Ind. Eng. Chem.*, May, 1931, V. 23, No. 5, p. 534-8.—Methods and apparatus used in the work are described. Curves are plotted showing relationship between rates of calcination and temperatures. Calcination of limestone takes place in a very narrow zone which is phase boundary between calcium carbonate and calcium oxide. Rate at which this zone advances from outside to inside of piece was constant for any given temperature used. Most of resistance to heat transfer into piece appears to be in narrow zone of calcination, and not in layer of calcined material. A method for determining surface area of particles from calcination data is suggested. The range of particle size used was 2.5 to 8.5 cm. —ROY N. YOUNG

Can by-product carbon dioxide be used in "dry ice" manufacture? D. H. KILLEFER. *Rock Products*, March 28, 1931, V. 34, No. 7, p. 71.—Gases with so low a concentration of CO_2 as those from cement and lime kilns cannot be profitably used at present for making dry ice. Gases from fermentation may be used at cost of removing odors, which is less than cost of removing other gases than CO_2 from usual combustion gases. Gas from coke burned under specially designed boiler has been used, the heat furnishing power to purify and compress about one-third of CO_2 produced.—EDMUND SHAW

PROPERTIES OF CONCRETE

Weather resistance of concrete brick. *Zement* (Germany), April 16, 1931, V. 20, No. 16, p. 375.—To show durability and weather resistance of concrete, a monument is illustrated which was built in 1867 and was awarded

a prize at world's fair in Paris. No visible defects can be detected on this 64 year old structure.—A. E. BEITLICH

The modulus of elasticity for concrete in compression. R. DUTRON. *Summary Reports First I. C. for C. and R. C.*—Modulus of elasticity for concrete is function of its strength, so by implication it depends upon all of factors which affect strength, richness in cement, nature and gauge of inert materials, fluidity, and age of concrete.—LA TECHNIQUE DES TRAVAUX

Study on slow deformations of cement. E. FREYSSINET. *Summary Reports First I. C. for C. and R. C.*—Study of the causes of error; provisional distinction between expansion and shrinkage; results of first series of experiments on rapid variations of shrinkage; variations of shrinkage during periods of long duration; influence of mechanical factors on shrinkage; thermo-hydrometric laws of shrinkage; hydro-elastic laws of shrinkage.—LA TECHNIQUE DES TRAVAUX

The design of the mix and pre-estimation of concrete.—The hardening of cements. L. VANDEPERRE. *Summary Reports First I. C. for C. and R. C.*—Inquiry as to mix of concrete with regard to materials available (cement, ballast or shingle, sand or dust) is based on previous determination of D (maximum diameter of the stones) and d (maximum diameter of the sand or dust). It has long been desired to transform curve of hardening of cements into a straight line by a logarithmic reflection of the values of the ages. In this hypothesis resistance of cement should be taken as increasing continually up to infinite age.—LA TECHNIQUE DES TRAVAUX

Study of the composition of concrete by means of modulus of fineness and of a water cement factor. ANT. BREBERA. *Summary Reports First I. C. for C. and R. C.*—The author considers "The provisional directions for the making of concrete," which he has arranged and which have been approved by the Ministry of Public Works of Czecho-Slovakia. According to these directions determination of best proportion of ballast and of sand is effected by a granulometric analysis on basis of ideal modulus of fineness, which he determines for materials of maximum determined size and by a mixture determined by diagram. Wetting of concrete corresponding to maximum resistance is deduced from author's original diagram.—LA TECHNIQUE DES TRAVAUX

Inert materials and the mechanical properties of concrete. R. DUTRON. *Summary Reports First I. C. for C. and R. C.*—Resistance of concrete to compression, tension, flexure and impact varies—apart from any ot er factor— with nature, shape and gradation of inert materials which constitute its skeleton. It may be stated that, in general, regarding inert materials dealt with in this paper, their order according to effect on compressive strength is almost the reverse of their order according to effect on the other resistances considered. —LA TECHNIQUE DES TRAVAUX

Sins of the concrete fellows. VAN DER KLOES. *Summary Reports First I. C. for C. and R. C.*—Inspection of a silo-building 10 to 12 yrs. old of armed concrete in a deplorable state of deterioration. The author attributes the bad condition to excess water in the concrete mixture, resulting in shrinking and bursting.—LA TECHNIQUE DES TRAVAUX

Mechanical theory of the setting and hardening of cement mortar and concrete. JEAN BASTA. *Summary Reports First I. C. for C. and R. C.*— In this theory, from the results of experimental tests with help of natural laws, are deduced rational equations for the resistance of concrete and also for their use in the calculations for constructions in concrete and reinforced concrete considered from both the static and dynamic points of view.—LA TECHNIQUE DES TRAVAUX

The use of concrete and ferro-concrete in the colonies. C. WOLTERBEEK. *Summary Reports First I. C. for C. and R. C.*—Suitability of concrete for construction in the colonies is discussed. Prevalence of good aggregates,

especially sand is of great importance. It is necessary to design watertight concrete without voids, especially in tropical countries, for great heat and blowing rain have the tendency to deteriorate the alkalinity of the concrete, and this results in a danger of rusting of embedded bars.—LA TECHNIQUE DES TRAVAUX

Light-weight concrete in practice. A. WEISS. *Zement* (Germany), Jan. 29, 1931, V. 20, No. 5, p. 101-5.—In an historical development of reinforced concrete skeleton construction method application of light-weight concrete for such structures is emphasized. Properties of this material are discussed and experiences with various kinds of light-weight concrete are described. Saving of space and high insulating properties are chief advantages.—A. E. BEITLICH

Concrete compressive strength and concrete age. RICHARD FAERBER. *Zement* (Germany), Feb. 19, 1931, V. 20, No. 8, p. 177-8.—A graphical method enables estimates of compressive strength of concrete at later ages. Two curves show relation between strength expressed in per cent of 28-day compressive strength and age in hours (in logarithmic scale) and days. Curves depend upon laboratory test results. Separate curves must be plotted for each individual cement. Method is of value in practice, especially for determination of strength of failed concrete and in cases where existing structures are enlarged or submitted to greater loads.—A. E. BEITLICH

Standard strength and concrete strength. WILHELM STOY. *Zement* (Germany), Feb. 26, 1931, V. 20, No. 9, p. 194-7.—Experiments were undertaken with 10 standard portland cements, 3 high quality portland cements and 3 natural cements to study relations between cement strength and concrete strength with specimens of varying consistency. Results of test support demands that in certain cases properties of cement should also be studied in mixtures of soft and wet consistency (representing better the conditions of practice) besides normal tests with earth-moist consistency. Sand of suitable grain composition should be used for these tests. It is also advisable to use a shaking table of smaller dimensions than asked for in standard specifications.—A. E. BEITLICH

Calculation of heat reactions in winter concreting. MAX MAYER. *Zement* (Germany), April 2, 1931, V. 20, No. 14, p. 322-5.—An empirical calculation method is derived which is based on 4 successive processes in mixing and placing operation of concrete. (1) Sand, gravel, cement and water of temperatures a, b, c and d are mixed and give a concrete of temperature e, (2) Concrete is transported to mold where it arrives with a temperature f, (3) is poured and cools off to temperature g. (4) After several hours concrete will have temperature h. Equations show final temperature of mixture of raw materials calculated from measured temperatures, heat capacities and radiation. Generation of heat in concrete due to chemical reactions of cement is important.—A. E. BEITLICH

About the elastic behavior of concrete with special consideration of diagonal tension. "Uber das elastische Verhalten von Beton mit besonderer Berucksichtigung der Querdehnung." HIROHIKO YOSHIDA. 1930, Julius Springer, Berlin (Germany), R. M. 11:00. Reviewed in *Zement* (Germany), March 12, 1931, V. 20, No. 11, p. 250.—Available information about determination of coefficient of transverse expansion of concrete is greatly enlarged by this book. After discussing previous work, different test methods are characterized such as compressive, tensile and bending test. Results show relations between stresses and elastic behavior of concrete under various loads. Effect of age, W/C ratio, normal compressive strength and cement content are described.—A. E. BEITLICH

New fundamental principles of concrete composition. "Neue Grundlagen der Betonzusammensetzung." OTTOKAR STERN, 1930, Editor: *Ostr. Ingenieur und Architekten Verein*, Vienna (Austria), R. M. 2.00. Reviewed in *Zement* (Germany), April 9, 1931, V. 20, No. 15, p. 352.—Preliminary work of a research program for numerous open questions of influence of aggregates

on quality and economy of concrete, discusses 2 important recent researches: Abrams' fineness modulus and Spindel's constructive concrete calculation. Complicated relations between concrete composition and concrete properties such as consistency, workability, density and strength properties are explained. Final conclusions are based on these theories and their importance for construction practice. Efforts are made to express numerically degree of workability and degree of consistency of concrete.—A. E. BEITLICH

Effect of heating on concrete materials in winter construction jobs. LUZ DAVID. *Zement* (Germany), March 5, 1931, V. 20, No. 10, p. 218-21.—Discussing work done by W. H. Bachelder (cf. *Eng. News-Rec.*, Dec. 18, 1930, p. 973) the author concludes: No objections exist against use of mixing water of temperatures of 140 to 176° F. when materials are mixed in right order. Strength of concrete at about 142° F. decreases only from 20 to 30 per cent. Favorable grain composition and use of high quality cements give in general strengths above specifications.—A. E. BEITLICH

Differentiation of the properties of cements. STIG GIERTZ-HEDSTROEM AND DONOVAN WERNER. *Zement* (Germany), March 5, 1931, V. 20, No. 10, p. 213-5.—After considering major properties of cements such as tensile and compressive strengths, solubility, shrinkage, heat development and setting, authors classify cement into 2 types: (1) Cement for hydraulic structures, which has decreased solubility and small heat development. This cement has probably greater shrinkage, smaller strength properties and slow hardening. (2) Cement for superstructures which hardens more rapidly and shows small shrinkage. It has higher strength, develops more heat and has a greater solubility.—A. E. BEITLICH

A method for the manufacture of concrete with especially high strength. WALTER MARSCHNER. *Zement* (Germany), April 2, 1931. V. 20, No. 14, p. 330.—In connection with a recent German application for patent (No. G 72951) Richard Grün uses as aggregates for manufacture of concrete entirely or partly broken portland cement clinker, high alumina cement clinker or an especially prepared burned material with a relatively high silica content. Compressive and tensile strength are greatly increased and durability in magnesium sulphate solution is considerably improved. Clinker reacts with cement only on its surface and its interior is not attacked even after long exposure to water. Tensile strength increases from 2 to 3 times under normal storage conditions and about 5 times in magnesium sulphate solution; compressive strength is about 3 times higher with addition of these materials than without them.—A. E. BEITLICH

Effect of limestone aggregates on strength of concrete and mortar. P. FILOSSOFOW AND A. SCHTEPETOW. *Zement* (Germany), Feb. 12, 1931, V. 20, No. 7, p. 152-5.—Compressive strength of various concrete mixtures made of high grade aggregates and limestone aggregates with different consistencies were examined. Strength of concrete varied with strength of coarse aggregates. Additions of aggregates of less strength affect strength of rich concrete more than of leaner mixtures. Strength of mortar, which corresponds to a concrete with strong aggregates, is less than strength of these concretes. Strength of mortar, corresponding to concretes with aggregates of less strength, is greater than strength of rich concrete (1:2:4) and less than that of lean mixtures (1:4:8). Poor aggregates affect mortar strength less than concrete strength. In lean mixes mortar strength and concrete strength are similar.—A. E. BEITLICH

Acoustic tests of materials of construction. *Genie Civil* (France), March 7, 1931, No. 10, p. 246-247.—In tests conducted at instigation of Touring Club of France at laboratory of Conservatory of Arts and Crafts, with M. Cellerier in charge, physical and mechanical properties of materials were investigated in their relation to transmission of sound. A distinction is made in acoustic tests between sounds, of given frequency and amplitude, and noises, which are the complex effect of many simple sounds. Readings of the ammeter

of the measuring appratus were recorded for the sound produced: i upon passing through sample section; and I upon direct reception. The ratio of transmitted sound intensity to direct sound intensity was recorded for each material. Parallel tests were made of heat conductivity, permeability, combustibility, hygroscopicity, porosity and strength.—M. A. CORBIN

Modern proportioning of concrete. V. BAEHRNER. *Zement* (Germany), Jan. 22, 1931, V. 20, No. 4, p. 82-4.—Research work in laboratory of Cementa cement plant in Malmo (Sweden) yielded a new concrete proportioning system which takes into consideration quality of cement, consistency of concrete and gradation of aggregates. For calculation of mixture must be known: desired strength of concrete, quality of cement, maximum size of aggregates and desired consistency. With these data and aid of W/C table and proportioning table, can be easily determined proportions of cement, aggregates and water. Water-cement ratio table shows relation between W/C and concrete strength made of cements of different quality. Proportioning table shows relation between W/C, cement-aggregate ratio, amount of cement per unit of volume of concrete and consistency of concrete.—A. E. BEITLICH

How concrete in Wacker Drive attained great strength. *Concrete* April, 1931, V. 38, No. 4, p. 15-16.—Strength of 11,400 lb. was attained in Wacker Drive concrete, Chicago, due to complete hydration of cement particles. Process of hydration is best understood if fineness, or size distribution, of cement is considered, particularly with respect to quantity of fine sizes present. Length of time required for hydration is shortened as size of cement particles decreases. Ordinarily in building construction work, only three of fine sizes, representing about 45 per cent of quantity of cement, have time to become completely hydrated during short curing period and before remaining water is dried out. Three larger sizes, or 55 per cent of total quantity are never hydrated. In outdoor concrete, subsequent periods of rainy weather supply moisture that causes resumption of hydration of coarser particles and eventually all cement becomes hydrated and further strength is attained. Higher early strength is obtained with ordinary portland cement, partly by greater fineness and increasing percentage of smaller particles.—C. BACHMANN

Researches in concrete aid the engineer. H. F. GONNERMAN. *Civil Engineering*, May, 1931, V. 1, No. 8, p. 727-732.—Reinforced concrete construction was first proposed about 70 years ago by E. Coignet, a French engineer, and first investigations were made from 1870 to 1880 by an American engineer, Thaddeus Hyatt. From 1880 to 1900 important contributions to knowledge of subject were made by Feret, Considere, Monier, Hennelbique, Christophe, Bauschinger, Tetmajer, Bach, Martens, Morsch and others. During the period 1900 to 1914 American investigators, Talbot, Hatt, Humphrey, Turneaure and Withey reported numerous researches but properties of plain concrete were given relatively little attention. Agencies now engaged in study of concrete and related materials include national technical organizations, government bureaus, industrial organizations, and colleges. One interesting investigation consists of a study of the behavior of a variety of concrete structures over a wide range of climatic exposures and particularly those in direct contact with water. Observations indicate principal destructive agents are repeated freezing and thawing of saturated, porous concrete and solvent action of water finding its way through porous concrete mass.—D. E. LARSON

Some notes on proportioning concrete. HAROLD JAMES VOGAN. *Trans. Institution Engineers Australia*, V. 12, 1931, p. 1-15.—Concrete specifications of 1920 and 1929 are compared and revolutionary changes in design of mixes discussed, with comprehensive resume of theories for concrete design. The author points out the great value of fundamental work by Feret on mortar mixes in 1892. Fuller's mechanical analysis method, Abrams' water-cement ratio method and McMillan's recent work are reviewed. The value and significance of Power's and Edwards' contributions have not been overlooked. The results of work of Talbot and Richart in 1923, which contains investigations of funda-

mental importance, enables correct interpretation of results of other investigators. General comparison of methods and theories reveals agreement by investigators on fundamentals not apparent upon casual examination. These findings included: Importance of voids in concrete or mortar and percentage of cement; that strength and density of concrete are affected by variation in size of sand particles more than by variation in stone particles. None of the experimenters showed any advantage gained from sand with sharp grains. Tests bearing on these theories have been conducted at Peter Nicol Russell School of Engineering, University of Sydney. Comparison of actual and calculated values of W/C shows differences ranging from +1% to +12% and from -7% to -34%. More experiments are in progress.—N. H. ROY

Basic principles of concrete making. F. R. MCMILLAN. *Civil Engineering*, April, 1931, V. 1, No. 7, p. 601-606.—When aggregate in concrete is completely surrounded by cement-water paste, forming a uniform and homogeneous mass, properties of the concrete are principally determined by the properties of the hardened paste. In some cases, the properties of concrete are greatly affected by certain characteristics of the aggregate, but this does not reduce the importance of the paste for, regardless of kind or combination of aggregate, any change in properties of paste will at once be reflected in character of concrete. Chemical reactions responsible for setting and hardening which transform plastic cement-water paste into hard binding medium are termed hydrolysis and hydration. Curing conditions required for completion of these reactions are time, favorable temperature, and continued presence of water. Hydration is essentially complete at from 6 to 12 months both for standard portland cements and for high early strength cements. Strengths are correspondingly lower for shorter periods. Factors affecting these strengths are: relative proportions of cement and water, or w/c ratio; completeness of chemical combination between cement and water, that is, curing; characteristics of cement. These three factors also determine other qualities of the hardened paste such as watertightness and durability. In general, watertightness is a function of w/c ratio, time and rate of hardening just as compressive strength is a function of same factors. Principal disintegrating agencies are repeated freezing and thawing and action of percolating water.—D. E. LARSON.

Aggregates and the physical properties of concretes. (See MATERIALS —AGGREGATES.)

ENGINEERING DESIGN

BRIDGES

The bridge of Prince Joseph Poniatowski over the Vistula in Warsaw. B. PLEBINSKI. *Summary Reports First I. C. for C. and R. C.*—This bridge is known in Polish history of bridge construction not only as the largest bridge in Poland, 9840 ft. long (including the viaducts), but chiefly because for the first time concrete has been used on a large scale for its construction together with iron and steel.—LA TECHNIQUE DES TRAVAUX

Railway underbridges of concrete and reinforced concrete in Spain. J. EUGENE RIBERA. *Summary Reports First I. C. for C. and R. C.*—Note contains complete information on very large number of underbridges in concrete and reinforced concrete constructed in Spain in a period of over 20 years, with drawings of bridges of Torre Montalvo (concrete arches of 100 ft.) and straight bays at a sharp skew with a wide span of 65 ft. for the transpyrenean railway from Lerida to Saint-Girons.—LE TECHNIQUE TRAVAUX

Reinforced concrete in engineering. M. GERARD. *Summary Reports First I. C. for C. and R. C.*—Author deals briefly with some classes of engineering structures to which reinforced concrete has been applied in Great Britain. First among these are bridges, of which many hundreds have been built on the Hennebique system since the year 1900. Three improvements in jetty, pier

and quay construction introduced by Mouchel and his successors are dealt with in the succeeding section of report.—LA TECHNIQUE DES TRAVAUX

Test measurements on the Arlington Memorial bridge at Washington, D. C. J. FISHBURN. *Summary Reports First I. C. for C. and R. C.*—The Arlington Memorial Bridge Commission and the National Bureau of Standards of the United States Department of Commerce are jointly conducting a series of tests on the Arlington Memorial bridge, across the Potomac River at Washington, D. C. to obtain data on structural behavior of one of arch spans. The paper discusses principally design and construction of instruments used and accuracy of data obtained. Change in bending strain at points along the arch extrados, which occurred between adjacent tests is shown as computed from the test data.—LA TECHNIQUE DES TRAVAUX

The possibility of substitution of concrete arches over very large spans for metallic structures of all kinds. E. FREYSSINET. *Summary Reports First I. C. for C. and R. C.*—First part, study of possibilities arising from properties of materials. Second part, choice of forms of structures. Third part, methods of carrying out work.—LA TECHNIQUE DES TRAVAUX

Note on arched bridges in Switzerland. R. MAILLART. *Summary Reports First I. C. for C. and R. C.*—As regards Switzerland the picture is completed by several works of native conception absolutely free of foreign influence. Whilst greater part of these have been described, principally in *Schweizerische Bauzeitung*, and in the Reinforced Concrete Handbook of Emperger, a new process of arch construction has been described in detail as particulars have not previously been published.—LA TECHNIQUE DES TRAVAUX

Recent developments in arch design. W. L. SCOTT. *Concrete Constr. Eng.* (England), March, 1931, V. 26, No. 3, p. 180-187.—Increase in spans of reinforced concrete bridges during recent years has led to investigation of variable factors which, in past have been ignored or assumed to vary in a manner amenable to straightforward mathematical treatment. Assumptions for moderate span bridges, up to 150 ft., are not sufficient for spans of several hundred feet. Approximate assumptions have been made to standardize influence lines. Principal assumption consists in considering axis of arch to coincide with line of resistance from total dead load and to be a parabola of second degree. This is only true for uniform dead load and for large spans. Error due to this assumption cannot be disregarded especially for arches of the solid spandrel type. Another assumption is that variation in the moment of inertia due to diminishing depth of arch towards the crown varies according to value $I = I_c / \cos \alpha$, where I_c = cross sectional inertia at crown and α = angle of inclination of arch ribs with horizontal. These two assumptions are permissible for moderate spans. However total dead load is not uniform, variation being greater with solid spandrel than open spandrel type arch bridges. Hence instead of taking the curve of the arch axis to be a "square" parabola it is necessary to find curve that will coincide with line of pressure for loading, part of which is uniform and part variable. Further refinement of arch curve is to consider bridge loaded with one-half of average distributed superimposed loading and to make arch axis curve fit this condition. This curve will be a parabola of the fourth degree and may be expressed as follows: $\frac{y}{f} = 1 - (1 - k)\, m^2 - km^4$ where y = vertical ordinate of any section to mean fibre above springing level, f = rise of arch above springing level, m = horizontal distance of sections from the center line of the arch in terms of the half span, and k = a constant depending on the load variation. For economical variation in moment of inertia it is found by practice that suitable depth at springing is about 1½ times that at crown, corresponding moment of inertia is about 3½ times that at crown. With object of standardizing the investigation following expression for inertia variation is given: $I = I_c / \cos \alpha\ (1 - 0.7m^4)$. Maximum negative and positive moment at a section are obtained

from influence lines using ½ calculated superload as covering entire bridge. Secondary thrust is permanently eccentric producing shortening and hence flattening of arch. Arch shortening is produced also by shrinkage of concrete, displacement of abutments, and drop in temperature. Two devices are used to eliminate secondary effects: temporary hinges or hydraulic jacks to open arch at crown.—J. MARIN

The strengthening of metallic latt.ces by reinforced concrete. FR. EMPERGER. *Summary Reports First I. C. for C. and R. C.*—Strengthening of metallic lattices is so carried out that compressed bars are covered with bound concrete and tension bars are reinforced by additional bars welded at their ends to lattice framework. The method is explained with aid of example of a cantilever girder with a central span of 36.0 metres.—LA TECHNIQUE DES TRAVAUX

Fundamental data for the calculation of solid bridges. "Berechnungsgrundlagen fur massive Brucken." 1930, Wilhelm Ernst and Son, Berlin (Germany), R. M. 1.50. Reviewed in *Zement* (Germany), March 12, 1931, V. 20, No. 11, p. 250.—Book contains a list of all necessary data in concise form for design of solid bridges. Special consideration is given to German standard specification.—A. E. BEITLICH

Bridge piers in Suisan Bay supply test of concreting methods. C. R. HARDING. *Concrete*, April, 1931, V. 38, No. 4, p. 9-11.—Bridge of Southern Pacific Ry., has a total length of 5603 ft. and comprises numerous spans of steel construction, to support which, concrete substructures were built with bases resting upon solid rock. These abutments and piers were designed to carry their own weight, that of the substructure and certain specified engine and train loads, critical combinations were made of maximum live-load reactions, total dead load of superstructure and substructure with and without allowance for bouyancy, together with overturning effects of wind pressure, acting longitudinally, and transversely on spans and piers, horizontal tractive forces from stopping and starting trains, and horizontal forces against piers from wave action and pressure from stream currents. To prevent shattering of the concrete by earthquake tremors, the main piers were heavily reinforced. —N. H. ROY

Design of a bridge over Lake Maeler in Stockholm (Sweden). RICHARD FAERBER. *Zement* (Germany), Jan. 29, 1931, V. 20, No. 5, p. 105-9.—Bridge design by Faerber and Schuhmacher, submitted to international contest (cf. *Zement* (Germany), July 1930, V. 19, No. 30, p. 712-4; *Journal A. C. I.*, Oct., 1930, V. 2, No. 2, *Abstr.* p. 39) embodies unique features. Use of rammed concrete is proposed. Design of 2 dissimilar arches was necessary on account of location of suitable sound rock for foundation of river pier. Spans of arches are 738.2 and 524.9 ft. and their heights are 74.1 and 66.9 ft. These are record dimensions for this type of bridge. Arches are 7.2 ft. thick at center and 12.1 ft. at abutments. Total width of bridge is 80.7 ft. Greatest stability and durability and smallest costs of upkeep were demanded. Arches are designed in 3 parallel strips to reduce costs of centerings. Detail are given of calculation of stresses and deformations of structure. Mixing devices are provided with automatic measuring equipment for water. Concrete is to be rammed by means of compressed air to increase strength and uniformity. Roadway consists of a number of similar slab sections, which rest on 4 horizontal beams, supported by 2 transverse beams.—A. E. BEITLICH

Method of arch design. G. P. MANNING. *Concrete Construction Eng.* (England), April, 1931, V. 26, No. 4, p. 250-53.—By using "ready made" influence lines labor of preliminary arch designs and estimates is reduced and errors eliminated. If abutments are expensive, open-spandrel arch is selected; if head room is insufficient ribs may be carried up through deck. For open-spandrels, spacing of ribs is next decided and deck designed. Two profiles are selected as being generally useful as first assumption. Influence lines are

obtained from tables given also maximum moments, thrusts, and shears for all sections of arch rib. Best shape of arch is determined from loads and working stresses for each particular case.—JOSEPH MARIN

A concrete arch of 460-ft. span. G. S. RICHARDSON. *Eng. News Record*, Apr. 23, 1931, V. 106, No. 17, p. 680-683.—The largest concrete bridge project yet planned by Allegheny County involves 1,500 ft. of high-level structure over Turtle Creek Valley and includes five reinforced concrete arches, one of which is 460 ft. long between pier centers. All piers and abutments are founded on solid rock. Each span has two arch ribs 14 ft. wide. Deep ribs were avoided on account of temperature and rib shortening stresses. All ribs are designed for maximum working stress of 800 lb. per sq. in. under combined dead load, live and impact load, rib-shortening and temperature stresses, this working stress being based on minimum 2000-lb. concrete. Temperature stresses are calculated for drop of 40 deg. and rise of 20 deg. F. Columns on the ribs are arranged in pairs so that, in cross-section, each bent forms a four-legged rigid frame with adequate lateral stiffness to tie two ribs together. Expansion joints are provided at the ends of span only.—D. E. LARSON

The contest for the Dreirosen bridge in Basel (Switzerland). *Zement* (Germany), March 26, 1931, V. 20, No. 13, p. 298-302.—In addition to description of bridges given by Berger (cf. *Zement* (Germany), Jan. 15, 1931, V. 20, No. 3, p. 59-61; *Journal A. C. I.*, April 1931, V. 2, No. 8, *Abstracts*, p. 223), two designs, awarded prices, are illustrated. (1) Continuous reinforced concrete beam with one fixed end at shore abutment and movable bearings at other supports. Bridge has 4 openings and rests on 2 river piers and 1 shore pier. Caisson method is proposed for placing of river piers. Calculation of stresses is given as well as exact data concerning excavating, proportioning of concrete and arrangement of mixing plant and transportation of concrete at site of construction. (2) Three-hinged reinforced concrete frame structure has 3 openings, 161.4, 331.4 and 161.4 ft. wide. Cross-section of 3 hollow beams which support roadway have interesting design. Roadway consists of cross-reinforced slab 8 in. thick. River pier foundations are caissons.—A. E. BEITLICH

The Echelsbach bridge. A Melan-arch with longest span. "Die Echelsbacher Brucke. Der weitestgespannte Melanbogen." F. DULL AND R. GERHART, 1931, Wilhelm Ernst und Sohn, Berlin (Germany), 76 p., 88 illustrations, R. M. 8.40. Reviewed in *Bautechnik* (Germany), March 20, 1931, V. 9, No. 13, p. 191.—A very complete picture of outstanding reinforced concrete arch bridge, great span of which was built without any centerings, is presented and various designs are discussed. Every phase of construction program is described in all details. Arrangement of site of construction, concrete mixing plant and transportation of structural materials are illustrated. Static calculations are given in full.—A. E. BEITLICH

Laboratory tests of reinforced concrete arches with decks. WILBUR M. WILSON. *U. of I. Eng. Exp. Sta. Bulletin* No. 226. April, 1931.—Eight reinforced concrete arches of 17 ft. 6 in. span and 4 ft. rise were tested to destruction. Two specimens were composed of ribs only. Two had rectangular deck slab supported on columns well above crown of arch, and had no expansion joints. Two had T-shaped deck slab supported on columns well above crown, one of these having expansion joints at third points. Two had T-shaped deck slab placed low and built monolithic with rib at crown, one of these having expansion joints at third points. All specimens were subjected to same distribution and intensity of loading (to failure) with greater intensity on one side of the center than on the other. Strains were measured on both concrete and reinforcing. Celluloid models of the experimental arches, and of other arches having various geometrical properties, were tested. Conclusions from this investigation are: Deck and arch rib act together structurally and analysis should be made on this basis for any important structure; expansion joints reduce the strengtening effect of the deck; when the deck is placed high above the rib at center (generally an unwarranted arrangement) expansion

joints may be necessary in medium and long spans; when vertical clearance is large it will generally be advantageous to increase rise of rib and place deck low with respect to rib.—ARTHUR R. LORD

Concrete trusses and 47-ft. slabs in new Seattle viaduct. CLARK H. ELDRIDGE, *Eng. News Record*, April 16, 1931, V. 106, No. 16, p. 642-644. Distinguishing features of reinforced-concrete design, of West Garfield St. bridge, recently completed in Seattle include heavy flat slab construction on spans up to 47 ft. and reinforced concrete trusses 60 ft. long. The structure is 3,010 ft. long between abutments. Design specifications were based on a live load of two 20-ton trucks with 100 per cent impact allowance on rear axle of one truck. A concrete mix of approximately 1:2:3, developing 3000 lb. per sq. in. in 28 days, was specified, and a working stress of 1,200 lb. per sq. in. used in design. The one-way slab section of bridge consists of bents of two or more columns capped with wide shallow beams. A heavy slab, reinforced primarily in one direction and thickened for negative moment at supports, spans directly from bent to bent, without beams or girders. Advantages of this type of construction include: simple formwork; large and unerowded reinforcing bars; use of concrete of low water content; thorough curing; maximum clearance; distribution of heavy wheel loads; low cost. Reinforced concrete trusses have 60-ft. spans with 15-ft. cantilever panels at both ends. A 13-ft. suspended slab spans between cantilever ends of adjacent trusses. Lower chords and tension diagonals of truss consist of flat structural-steel bars connecting with ends of compression members by curved plates that loop around and engage concrete. These flat bars are wrapped with wire mesh and concrete is poured around the assembly.—D. E. LARSON

BUILDINGS

Applications of reinforced concrete in building construction. A. LOPEZ-FRANCO. *Summary Reports First I. C. for C. and R. C.*—A summary of the works of this kind planned or carried out by authors snce 1910 in which they began to practice in this speciality, omitting details of procedure and design.—LA TECHNIQUE DES TRAVAUX

Roof of thin reinforced concrete slabs.—System Zeiss-Dywidag. FR. DISCHINGER. *Summary Reports First I. C. for C. and R. C.*—Author gives complete description of development of roof of thin reinforced concrete slabs of Zeiss-Dywidag system, which are known by monumental structures of covered halls of Frankfort and Leipzig. He examines the possibilities of further developments.—LA TECHNIQUE DES TRAVAUX

Spacious reinforced concrete hangar for the Seville airport. A. PENA BOEUF. *Summary Reports First I. C. for C. and R. C.*—This hangar is destined to hold two Zeppelin type airships. It is designed to be built of reinforced concrete, forming a series of parabolic arches, the centre of gravity of which lies on the directrix of main arch which is in shape of a normal catenary. It is calculated for the fundamental effects: dead weight, force of wind and thermic effect, and two hypotheses have been taken, corresponding to the thrust of the outer air and the simultaneous thrust of interior.—LA TECHNIQUE TRAVAUX

State hospital for children in Bern (Switzerland). F. HILLER. *Schweizerische Bauzeitung* (Switzerland), Jan. 3, 1931, V. 97, No. 1, p. 3-6.—Modern hospital was erected in reinforced concrete skeleton type. All floors, except cantilevered rest room floors, were built as supportless hollow brick structures. Floors of rest rooms were made of solid reinforced concrete. Visible expansion joints allow for shrinkage of concrete.—A. E. BEITLICH

Concrete mill design governed by machinery layout. EDMUND WILKES, JR., *Eng. News Record*, Apr. 2, 1931, V. 106, No. 14, p. 561-564.—Plant of the Ralston-Purina Co., at Denver, Colo., offers an interesting example how predetermined machinery layouts may affect reinforced concrete design.

Three main units of plant incorporated in a single structure are (1) a grain elevator headhouse 56 x 76 ft. in plan and 112 ft. high, surmounted by a cupola 74 ft. high; (2) a battery of fifteen grain tanks 18 ft. in diameter and 89 ft. high; (3) a feed mill 52 x 180 ft. in plan and eight stories high. Feed mill building contains five groups of rectangular bins nearly all of which are two stories deep. Bin locations were fixed by machinery layout and could not be shifted to accommodate structural design. Room for beams underneath bin walls was not available so the walls were made self supporting and concrete floors were suspended from them. Columns, beams and walls in bin area were constructed of reinforced concrete poured in continuous vertical run using sliding forms. Pockets were left in columns and walls to support floors, which were designed as two-way slabs supported by beams in both directions with exception of those supported from walls.—D. E. LARSON

Results of measurements of deformations and stresses of mushroom slabs. M. ROS AND A. EICHINGER, *First I. C. for C. and R. C.*, Liege, Sept., 1930.—Tests were made on six different mushroom slabs having square panels. Results of measurements of stresses, for Poisson's ratio $m = \infty$ and $m = 6$, and deflections are summarized for two buildings as follows: (1) Maximum moments in middle of panel, or half way between two columns are not essentially different from one another for loads uniformly distributed in a series of panels in line. (2) Points of inflection of elastic curve, for plane passing through axis of columns, i. e., the points where moment is zero, are from $0.22L$ to $0.28L$ distant from the axis of columns. (3) Bending moment of columns below and above slab (M_o and M_u), the latter having load uniformly distributed in a series of panels in line, permit: $M = M_o + M_u = \frac{pl^3}{16.8}$. M should be distributed on the two columns. For unusually strong columns and for considerably developed mushrooms, bending moment which obtains at two columns, reaches a value perceptibly higher, and may even exceed the value, corresponding to entirely built-in or fixed condition, $M = \frac{pl^3}{12}$. Maillart in 1908 found the following: for uniform load in panel of mushroom slab considered, calculated according to surface of deflection, in middle of panel, $M = \frac{Pl^2}{37}$, halfway between two columns, $M = \frac{Pl^2}{22}$. On two different mushroom slabs with ratio of length to breadth 1.8:10, results summarized are: (1) Bending moments in middle of panel and half way between two columns are equal for a load uniformly distributed in a series of panels in line (with longer sides adjacent) (2) Calculation by method of substitute or auxiliary frame gives values in close agreement with values obtained by measurements, for: positive moments in middle of a panel and halfway between two columns; negative moments at built-in or fixed point; bending moments of columns; and for deflections.—R. B. B. MOORMAN

DAMS

Hydro-electric plant at Porjus, Sweden. GEORGE WILLOCK. *Canadian Engineer*, Jan. 27, 1931, V. 60, No. 4, p. 9.—Hydro electric power plant, run and owned by Royal Board of Water Falls, Sweden, most northerly power plant in the world, has dam 4100 ft. long consisting of three parts, one portion forming the intake. To reduce ice pressure upstream faces are given a gradual slope with piers, walls, crest and downstream sides of spillways protected by a stone coating anchored to concrete foundations. The intake structure is of solid concrete.—G. M. WILLIAMS

Honeycomb gravity-type concrete dams. C. E. GRUNSKY. *Civil Engineering*, April, 1930, V. 1, No. 7, p. 630-631.—A new type of concrete gravity dam having galleries for cooling and draining the interior is suggested

by author. There is tendency for temperature in interior of massive concrete structures to rise for some time after concrete has been placed. To simplify the high gravity dam, it should be straight, not arched; to take advantage of water pressure, treated as load, its upstream face should be inclined; to prevent cracking there should be both horizontal and vertical joints, not too far apart; to keep water from permeating concrete and reaching joints and cracks where its presence under pressure would be undesirable, masonry near upstream face should be thoroughly drained, or a metal diaphragm should be provided. Any part of dam above an assumed horizontal plane at any elevation should be planned to satisfy the requirements of a dam with its base at level of assumed plane. The new dam suggested by author has an inclined upstream face with an imbedded metal membrane to prevent penetration of water. Fair-sized galleries, open at the downstream face, are provided both to cool the interior and reduce the weight of masonry. Vertical joints are carried from top of dam to foundation.—D. E. LARSON.

MISCELLANEOUS

The largest reinforced concrete syphons in the world. J. EUGENE RIBERA. *Summary Reports First I. C. for C. and R. C.*—Siphon of Sosa is formed of two tubes 3350 ft. long and 12.5 ft. in diameter; that of Albelda of a tube 2360 ft. long and 13.1 ft. in diameter. Both constant head of 85 and 98.5 ft. and for more than 20 years are of normal service to irrigate 247,000 acres in Aragon and Catalonia.—LA TECHNIQUE DES TRAVAUX

Reinforced concrete construction in Poland. W. PASZKOWSKI. *Summary Reports First I. C. for C. and R. C.*—Conditions in Poland have favored use and development of reinforced concrete from its introduction, the country being rich in raw materials required for making portland cement and in all kinds of stony materials suitable for concrete.—LA TECHNIQUE TRAVAUX

Progress of reinforced concrete construction in Germany. W. PETRY. *Summary Reports First I. C. for C. and R. C.*—The basis of the constructive design and carrying out of works in reinforced concrete is governed by official regulations, at the moment in process of revision. Working loads depend on execution and the supervision. The conditions of a good execution will be assured and increased by the directions of control of German Association of Reinforced Concrete.—LA TECHNIQUE DES TRAVAUX

Tests on the hooked ends of reinforcing bars. TH. WYSS. *Summary Reports First I. C. for C. and R. C.*—Question of maximum curvature permissible for anchorage hooks is discussed on basis of recent experiments on forming of steel reinforcement.—LA TECHNIQUE DES TRAVAUX

Railway sleepers in reinforced concrete. FR. EMPERGER. *Summary Reports First I. C. for C. and R. C.*—Two slabs (blocks) and a junction piece are assembled in position for a single sleeper. Slabs are of a form appropriate to their use and suitable for bearing on ballast. A complete comparison is made with usual systems of reinforced concrete sleepers in use on the Hungarian and French railways.—LA TECHNIQUE DES TRAVAUX

Theorical and experimental study of the load capacity of concrete piles.—Concrete piles with enlarged base. J. SPRENGER. *Summary Reports First I. C. for C. and R. C.*—The "reinforced concrete pile with enlarged base," an invention of the author, proved to have larger resistance at base than an ordinary concrete pile. For this reason an increase of 50 per cent to the allowable load seems to be correct. A formula for reinforced concrete piles is deducted.—LA TECHNIQUE DES TRAVAUX

The crippling of rectilinear members of eccentric loads. M. R. CHAMBAUD. *Summary Reports First I. C. for C. and R. C.*—The analysis of Euler which determines exactly the critical crippling load for axial loading cannot be extended to crippling by eccentric loading, and attempts made in this direction have tended to some extent to falsify ideas on the true nature of

the phenomenon. Present memoire presents an exact theory of eccentric crippling based on complete equations of bending and properties of elliptic functions.—LA TECHNIQUE DES TRAVAUX

Calculation of columns of reinforced concrete with spiral reinforcement. J. PODOLSKY. *Summary Reports First I. C. for C. and R. C.*—In the calculation for buckling of reinforced concrete columns with spiral reinforcement, limit of the resistance of the concrete is determined by formula of M. Navier. We can find by two methods the radius of inertia (radius of gyration) of section of columns reduced to the form of concrete. We can determine this radius of inertia. The two methods give satisfactory results.—LA TECHNIQUE DES TRAVAUX

Experimental research with influence lines relative to plane rigid constructions. G. MAGNEL. *Summary Reports First I. C. for C. and R. C.*—Structures in reinforced concrete present numerous problems in the solution of rigid systems. The most perfect systems of calculations are based on a series of simplifying hypothesis. Author's microinfluentiometer permits formation on celluloid models of influence lines of rigid elements with a maximum error of 2 per cent.—LA TECHNIQUE DES TRAVAUX

Calculation of combined bending, (bending and tension) by the aid of a new rule adapted from the logarithmic system of Rieger for the calculation of combined bending (bending and compression) by the aid of 3rd rule adapted to the rule of Rieger. J. RIEGER. *Summary Reports First I. C. for C. and R. C.*—The author has found it necessary to complete his rule by a fourth rule for the calculation of eccentric tension. The disposition of this rule is analogous to that of the 3rd rule.—LA TECHNIQUE DES TRAVAUX

Report on plane bending. JOHN SOUBRE. *Summary Reports First I. C. for C. and R. C.*—Author has deliberately confined his work to case in which section is subjected to compression throughout its extent. Taking as a point of reference the neutral axis and not centre of gravity of imaginary section, he states the various equations of equilibrium and derives from them a series of formulae which constitute an easy method of calculation applicable to various problems met with in current practice.—LA TECHNIQUE DES TRAVAUX

The mean fibre of large rigid arches. F. CAMPUS. *Summary Reports First I. C. for C. and R. C.*—By reason of permanent, elastic and reversible shortenings of large rigid arches, the funicular polygon of fixed loads does not constitute most favorable forms of mean fibre of these structures. A direct simple method of correction is deduced from the ingenious conception of Prof. E. Timochenko, which consists of calculating hyperstatic arches, of which the mean fibre coincides with the funicular polygon of the loads or is very near to it, by taking as a basic isostatic system the three hinged arches.—LA TECHNIQUE DES TRAVAUX

Experiments of the form of lines of principal stress in problems of two dimensions. AUGUSTE MESNAGER. *Summary Reports First I. C. for C. and R. C.*—It is known that in two dimensions lines of principal stress form two families of orthogonal curves which are perpendicular or tangent to contours, and that resultant angles form single points of first order and salient angles of second. By superimposing the two solutions we obtain forms relative to problems of St. Venant for girders loaded at one point.—LA TECHNIQUE DES TRAVAUX

Calculation of relatively thin annular sections uniformly reinforced on their circumference and stressed by plane bending and direct load. LOUIS BAES, LUCIEN VANDEPERRE AND JACQUES VERDEYEN. *Summary Reports First I. C. for C. and R. C.*—The problem of annular sections of this type is chiefly met in pylons or posts and chimneys in reinforced concrete. The authors have made the graphs which form the subject of the note in connection

with an expert investigation for a chimney of reinforced concrete.—LA TECHNIQUE DES TRAVAUX

Calculation of straight members of rectangular section doubly reinforced and stressed by plane bending and direct load. LOUIS BAES AND LUCIEN VANDEPERRE. *Summary Reports First I. C. for C. and R. C.*—The note presents two types of solutions by graphs. The first consists of a series of five graphs of which one only is given.—LA TECHNIQUE DES TRAVAUX

Examples of the construction statics. "Ubungsaufgaben aus der Baustatik." RICHARD LINDT, 1930, Max Jänecke, Leipzig (Germany), Part 1 and 2, RM. 3.00. Reviewed in *Bautechnik* (Germany), March 20, 1931, V. 9, No. 13, p. 195.—More than 540 examples from all fields of statics are given. They deal with composition and separation of forces, determination of loads and stresses in structural members, calculation of beams and columns, strength, wind pressure, water pressure and earth pressure. Laws of statics are illustrated.—A. E. BEITLICH

Symmetrically reinforced sections under eccentric loads. A. PUCHER. *Beton u. Eisen* (Germany), April 20, 1931, V. 30, Heft 8, p. 156-158.—In consideration of concrete stresses in extreme fibres of rectangular and ring sections eccentrically loaded, concrete symbols are substituted into formula for stress due to axial load combined with bending and an expression is obtained for the ratio of max. and the min, f_c to the average f_c in terms of properties of section, eccentricity, constant n (15 is used), and steel ratio. This expression is for construction of charts giving for various eccentricities and steel ratios the ratio of max. and min. f_c to average f_c. Chart for rectangular sections covers a range of (total) p values up to 6 per cent and eccentricities up to 0.35. Chart for ring sections includes sections varying from thin shelled cylinders (shell thickness equal to 0.025 of outside diameter) to solid sections. Charts are constructed on assumption that concrete takes tension and that n equals 15.—A. A. BRIELMAIER

Economic design of reinforced concrete piles. J. P. PORTER. *Concrete Constr. Eng.* (England), March, 1931, V. 26, No. 3, p. 202-210.—Greater economy in design of concrete piles is permitted because of development in strength of cement, introduction of rapid-hardening cement, greater care taken in making concrete and information on bearing values of piles. Factor of safety is determined based upon: quality of material passed through, ratio between bearing and frictional support, ratio between dead and live load and possibility of frictional support being increased by re-settlement of the ground. Second step is to determine dimensions and arrangement of the piles to be used. Considerations in designing a series of piles are: quality of concrete, stresses to be used, bending moments during handling, safe short-column length of pile and probable ground resistance. Allowable stress of 18,000 lb. per sq. in. in steel is proposed. Table given of allowable compressive strengths of concrete of different grades for a factor of safety of 3 and value of modulus ratio $m = 15$, 12, 10, and 8 for the different grades of concrete considered. Allowable stresses in shear for concrete is based on assumption of Navier's theory of failure. Attempt is made to treat subject of lateral reinforcement theoretically rather than empirically.—J. MARIN

The economic design of reinforced concrete piles. J. P. PORTER. *Concrete Constr. Eng.* (England), April, 1931, V. 26, No. 4, p. 236-44.—Loads and forces sustained by piles are: (1) bending and shear stresses while being handled, (2) compression, shear, and possibly buckling crushing stresses while being driven—in body of pile, (3) crushing stresses while being driven—in head and foot of pile, and (4) compression, shear and possibly buckling stresses under working load. Reinforcement of 2 and 4 per cent is selected. Expressions are given for maximum moments produced for specified types of slings, and for maximum length of piles in terms of resisting moment and cross section. Approximate relation is obtained for resisting moment in terms of

cross-sectional dimensions of pile, area of steel, distance of steel from edge and distance of neutral axis from outer fibre. Shear strength for long piles is supplied by links. Relation is deduced for shear assuming pile supported at one end and at quarter point. Shear steel is necessary to resist impact stresses, to give resilience, to resist bursting pressure of concrete and to prevent buckling. Quantity of shear steel should be at least doubled for the first 2 or 3 ft. at head and foot of pile. Pile is assumed as "short column" if hammer blow is axial and if equivalent "unsupported" length does not exceed 15 times core width. In selection of piles, number and spacing to be used is determined and working load per pile derived. Ultimate ground resistance to which pile is to be driven is determined. If "short column action" only is the condition, a suitable pile can be selected from table. If "long column action" is likely, ultimate ground resistance is multiplied by a reduction factor to determine the safe "short column" load.—JOSEPH MARIN

Modern reinforced concrete structures of special designs. WILHELM PETRY. *Zement* (Germany), Feb. 12, 1931, V. 20, No. 7-8, p. 149-52, 170-4.—Recently completed reinforced concrete structures of special interest on account of peculiarities in their design and construction include: Retaining wall at sand beach in Wannsee (Germany); reinforced concrete tunnel for bituminous coal mine in Finkenheerd which was constructed over railroad of mine, under considerable difficulties; foundation of a traveling coal unloading crane on Teltow channel; Several loading platforms for sand, lime-stone for new cement plant in Neuwied (Germany), coal and aggregates.—A. E. BEITLICH

The rational reinforcement of concrete. *Engineering* (England), March 27, 1931, V. 131, No. 3402, p. 409.—Distribution and arrangement of steel in secondary reinforcement to resist shearing stresses should provide for its disposal along tension lines of principal stress. Use of stirrups perpendicular to neutral axis of beam is uneconomical since principal stresses cut neutral axis at angle of 45 deg. Main reinforcing bars not needed for tension should never terminate parallel with the upper surface of beam but should first be turned at an angle of 45 deg. to neutral axis, and then bent round further so as to end at right angles to upper surface. Any shear can be resolved into a tension and a compression at right angles to each other. From symmetry best arrangement of metal to take tensions will be to place it at right angles with line of thrust. Hence in reinforced concrete truss steel work should be arranged horizontally in tension members and at right angles to axis of struts. In case of bow string girders these members are subject to bending moment as well as direct thrust or pulls and reinforcement should be given a corresponding curvature, meeting compression face perpendicularly but tension side at angle of 45 deg.—G. M. WILLIAMS

Report of the 33rd main meeting of the German Concrete Association on March 17, 18 and 19, 1930. "Bericht uber die 33ste Hauptversammlung des deutschen Beton-Vereins am 17., 18. und 19. Marz 1930." 510 pages with numerous illustrations and tables. Reviewed in *Bautechnik* (Germany), March 20, 1931, V. 9, No. 13, p. 193.—Book contains complete text of 15 papers presented at this meeting and gives a review of newest developments in reinforced concrete practice and its many applications. Descriptions of outstanding structures, mostly erected in Germany, and theoretical discussions are included.—A. E. BEITLICH

Load members. "Belastungsglieder. "A. KLEINLOGEL, 1931, Wilhelm Ernst und Sohn, Berlin (Germany), 4th edition, 117 pp., 127 illustrations, 32 tables, RM. 9.00. Reviewed in *Bautechnik* (Germany), March 20, 1931, V. 9, No. 13, p. 192.—This is a supplement to several books by same author. Besides load members it discusses all static values, moments and transverse stresses. Formulas and equations are presented in simple and clear manner. Main chapter deals with stresses and moments of simple beam for single loads, square, triangle and trapezoid-loads.—A. E. BEITLICH

Stirrups in concrete beams located with charts. T. GERMUNDSSON. *Concrete*, May, 1931, V. 38, No. 5, p. 41-42.—A large chart has been constructed, based upon usual method of determining spacing of stirrups and composed of a series of vertical lines for each size of bar commonly used. It is necessary to calculate total shear to be carried by stirrups at two points, at support and at center of span. A line connecting these two points on the chart enables designer to read directly number of stirrups required and their spacing. —N. H. ROY

Short-cuts in structural design. JAMES R. GRIFFITH. *Concrete*, April, 1931, V. 38, No. 4, p. 36-38.—Chart II gives intensity of pressure under footings. Chart resembles a contour map of a hill. It may be used in computing soil pressures under foundation walls of buildings, chimney footings, bridge piers and similar cases. To use the chart, overturning moments must be calculated. These moments are taken about base and parallel to either side of structure. Maximum pressure under footing is a constant, K_r, times the unit load on the base. The value K_r is read from the chart, using the overturning moment, the total load on the base and one dimension of the base.—N. H. ROY

Application of influence lines to the design of rigid frames. W. CHERRE. *Concrete Construction Eng.* (England) April, 1931, V. 26, No. 4, p. 252-64.—Method of constructing influence lines for moments and shears in rigid frames, is based upon assumed variation in moment of inertia expressed by an equation giving moment of inertia at any section in terms of that at center of span, length of span, ratios of inertias at center and at supports, and a coefficient. Expressions are derived for moments at ends of spans in terms of rotations at ends. Equations defining positions of fixed points are determined, from which rotations of girders and columns of frames are obtained. From values of rotations, bending moments due to unit loads for different sections are found and plotted. From these curves influence lines are drawn. Method of constructing influence lines for shear is outlined.—JOSEPH MARIN

Nomogram for calculating the allowable distance between vertical or inclined stirrups of beams and slabs as determined by diagonal tension and shear. JORGE QUIJANO. *Ingenieria* (Mexico), June, 1930, V. 4, No. 6, p. 253-254. A nomographic chart for use in designing or investigating the distances between vertical or inclined stirrups, which solves the relation $T = S\,(v—2.8)b$.—(Author's correction to 5th abstract, A. C. I JOURNAL, Jan. 1931; *Abstracts* V. 2, p. 146).

ROADS AND PAVEMENTS

Where more research is needed in concrete highway work. *Concrete* April, 1931, V. 38, No. 4, pp. 33-34.—Replies to inquiry among engineering officials of state highway departments indicate more information is needed on a number of problems, in approximately following order of importance: (1) Studies of subgrade soils, drainage and frost action, and their relation to concrete pavements. (2) Design of concrete mixtures, with special reference to simplicity of procedure. (3) Factors affecting durability of concrete. (4) Design of pavement slabs and sections, and studies of slab action. (5) Comparative efficiency of improved methods of curing. (6) Traffic surveys. (7) Concrete materials for highway pavements. (8) Use of coarser sizes of aggregates in concrete mixtures. (9) Construction methods. (10) Causes and prevention of cracking. (11) Studies of cement.—C. BACHMANN

Cement joints in concrete roads. C. H. CONNER. *Canadian Engineer*, March 31, 1931, V. 60, No. 13, p. 24.—Inspections show that concrete pavements with properly placed steel reinforcement, divided longitudinally and transversely by joints give far better service than plain concrete without joints. Generally all pavements 18 ft. or wider are divided by longitudinal joints. Some are cut through pavement and joint filled with filler while others may be hidden or partially cut through. Spacing of transverse joints usually varies between 30 and 50 ft. Present tendency is toward use of wide expansion joints

spaced at rather wide intervals with one or two contraction joints between them, such as ¾ in. expansion joint at 80-ft. intervals with one dummy contraction joint at 40 ft. intervals for a 40-ft. spacing. Generally a 1-in expansion interval for each 100 ft. of slab is recommended.—G. M. WILLIAMS

ARCHITECTURAL DESIGN

The new telegraph construction office building in Stuttgart (Germany). *Zement* (Germany), March 12, 1931, V. 20, No. 11, p. 247.—Large structure was built entirely of reinforced concrete. Building, 460 ft. long, is especially distinguished by precision in construction of exterior and interior. Architectural design is discussed.—A. E. BEITLICH

The architecture of reinforced concrete in Holland. J. DUIKER AND G. MEYERS. *Summary Reports First I. C. for C. and R. C.*—In Holland, as in other countries, reinforced concrete architecture has no existence as a characteristic national architecture. The architecture which is characteristic of reinforced concrete derives from the skeletal form of construction adopted; outside walls carry no load but serve only to separate the interior from the exterior. Such a way of building can allow a front wall almost entirely of glass. Style is determined by the principle which underlies construction; thus it may be noted how Classical and Gothic construction gradually acquired greater lightness and delicacy. It is in this way that reinforced concrete may lead to the development of a new style.—LA TECHNIQUE DES TRAVAUX

Decorative concrete. JOHN J. EARLEY. *Concrete Products*, April, 1931. V. 40, No. 4, p. 50-52.—In review of work done by the writer in developing use of color in concrete by means of colored aggregates, he states that recently a doubt has arisen not as to superiority of exposed aggregate method but as to universality of its application. Its use is likely to be restricted, leaving need for another treatment of concrete for more general use. This will perhaps take the form of a surface treatment though one that will be more nearly an integral part of the concrete itself than anything so far developed.—E. S. HANSON

The new reinforced concrete bridge with artificial granite surface over the Escaut at Eyne, Belgium. E. TASHJIAN. *Genie Civil*, (France) April 11, 1931, No. 15, p. 376-377.—Bridge built by State of Ohio in commemoration of the passing of the 37th Division of the American Army in October 1918, replaces structure torn down in 1914 by retreating allied armies. Its graceful lines, decorative sculptures and artificial granite surface are much admired. It was designed by Walker & Weeks, Architects, Cleveland, Ohio and erected by Messrs. De Cavel, Chief Engineer, Heylbroeck and De Nayer, Assistant Engineers of Roads and Bridges, Belgium. Artificial granite forms an integral part of structure, being poured into a space next to form, separated from the interior by a wire mesh, keeping in place special aggregate consisting of 40 per cent quartz, 40 per cent white Carrara marble, 20 per cent black Carrara marble. White cement was used.—M. A. CORBIN

FIELD CONSTRUCTION

BRIDGES

Arch bridges. "Bogenbruecken." C. KERSTEN, 1930, Wilhelm Ernst and Son, Berlin (Germany), second volume of: Reinforced concrete bridges "Bruecken aus Eisenbeton," 5th revised edition. R. M. 16.50. Reviewed in *Zement* (Germany), March 5, 1931, V. 20, No. 10, p. 226.—In a comprehensive treatise on construction of reinforced concrete arch bridges, numerous forms and systems are arranged according to different viewpoints. Examples of loading tests are of special value for consideration of questions of static. Construction and centerings are described besides presentation of static calculations.—A. E. BEITLICH

The use of pre-cast reinforced concrete slabs for bridge floors. RAFAEL CEBALLOS PARON. *Summary Reports First I. C. for C. and R. C.*—This

paper describes the arrangements for handling reinforced concrete slabs, precast in quantities off the site, for eventual use, suitably erected, to form the floor of a double track railway bridge of concrete.—LA TECHNIQUE DES TRAVAUX

Concrete trusses and 47-ft. flat slabs in new Seattle viaduct. CLARK H. ELDRIDGE. *Eng. News Record*, April 16, 1931, V. 106, No. 16, p. 642.—The West Garfield St. bridge recently completed in Seattle at a cost of about $750,000 includes heavy one-way flat slab construction on spans up to a maximum of 47 ft., and reinforced-concrete trusses 60 ft. long. Structure is 3010 ft. long, with a 36-ft. road and two 6-ft. walks. To meet design stress of 1200 lb. per sq. in., a 1:2:3 mix was used which attained an average 28-day strength of 3867 lb. per sq. in. Both trusses and deck were concreted in one operation.—GLENN MURPHY

Study of viaducts of variable span. G. GARAT. *Genie Civil*, (France) Jan. 31, 1931, No. 5, p. 109-114.—About 50 viaducts are under construction at present on the line Tanap—Thakhek, Indochina and consist of reinforced concrete spans resting on masonry supports. Spans vary from 19.7 ft. to 49.2 ft. Ease of construction and esthetic considerations have led to adoption of constant depth for all spans. Careful cost analysis including numerous diagrams representing cost per meter of the viaducts, showed that no appreciable cost increase was produced by adopting a constant depth.—M. A. CORBIN

Construction of a bridge over river Glan in Odernheim for highway between Homburg and Bingen (Germany). SEYWALD. *Zement* (Germany), Feb. 5, 1931, V. 20, No. 6, p. 125-9.—Old structure which had suffered by increased traffic during war time was replaced by reinforced concrete arch bridge. Sound underground was found in a depth of 20 ft. Span of single arch is 131.2 ft., roadway is 19.7 ft. wide and has a 4.1 ft. wide sidewalk on either side. Arch is provided with 3 lead hinges. Foundations were made in open excavations protected by Larsen steel sheet pilings. Centering is of normal design. Of special interest are expansion joints. A 1:4:8 concrete mix was used for abutments, 1:2:3 and 1:3:5:6.5 mixtures for arch and a 1:2:3 mix for roadway slab.—A. E. BEITLICH

Bridge piers in Suisun Bay supply test of concreting methods. C. R. HARDING. *Concrete*, April, 1931, V. 38, No. 4, p. 9-11.—To prevent shattering by earthquake, piers for railroad bridge, across Suisun Bay, at San Francisco, were heavily reinforced with steel rods. Concrete was controlled by W/C ratio method. Rock and sand were weighed, and cement, to which was added 3 per cent diatomaceous silica, was added from sacks. Adjustments were made for free water in aggregate. Concrete was mixed 90 seconds after all materials were in the mixer. Concrete mix, by weight, of 1:2.5:4.4, included 5 sacks of cement per cu. yd. and 6.75 gal. water per sack. Compressive strengths per sq. in. were 2,000 and 3,200 lb. at 7 and 28 da. Eight pedestal piers, located on land were built in wood forms, pier footings resting directly on bedrock. Mix was 1:2.42:4.04 by weight, with 5.46 sacks cement per cu. yd., and 6.25 gals. water per sack. Average compressive strengths were 1,760, 2,630, and 3,350 lb. at 7,14 and 28 da. Wood forms for 14 pedestal piers were built inside cofferdams and concrete placed in dry. Concrete for shafts was 1:2:3 mix by volume, with 6.49 sacks cement per cu. yd. of concrete, and 5.25 gal. water per sack of cement. Strengths were 2,700, 3,500 and 4,500 lb. per sq. in. at 7, 28, and 90 days.—C. BACHMANN

Reinforced concrete bridge with rigid reinforcement. *Genie Civil* (France), Feb. 7, 1931, No. 6, p. 139-141.—To reduce dead weight, bridge near Echelsbach, Bavaria, across the Ammer with a 426.5 ft. span, and rise of 104.3 ft. was designed as two arches of rectangular section, measuring 4.9 ft. in width and varying in depth from 6.56 ft. at crown to 10.5 ft. at skewback. These arches are spaced 19.7 ft. axis to axis and are hollow. Special feature of bridge is its reinforcement, which permitted erection without false work with independent

foundations. Bridge was erected cantilever fashion and was encased in concrete using light forms, this system of rigid reinforcement being known as Melan and Spangenberg system. Its application for a large span presented difficulty of ascertaining variable stresses produced by progressive concreting. Before concreting bridge was subjected to a load of gravel equivalent to total load it was finally to support, this load being removed as concreting progressed. —M. A. CORBIN

The Lorraine bridge over the river Aare in Bern (Switzerland). ROB MAILLARD. *Schweizerische Bauzeitung* (Switzerland), Jan. 3, 10, 17, 31, 1931, V. 97, No. 1, 2, 3 and 5, p. 1-3, 17-20, 23-6, 47-9.—Center arch of large concrete highway bridge has elliptical shape and span of 262.5 ft. Two circular arches with spans of 55.8 ft. support approaches on either side. Large concrete blocks, surfaces of which were treated by Contex method, were used. Abutments rest on sound rock. Roadway is provided with two cantilevered sidewalks, and is supported by 8 horizontal and 44 transverse concrete walls. Center and side arches are separated by expansion joints. Concrete mixing plants were located on either side of river. Centering of middle arch had unusual design. Placing of concrete blocks was started from both sides by cable crane. Data are furnished concerning physical tests of structural materials, laboratory tests, observations of movements in arch and temperature measurements.—A. E. BEITLICH

Repair of the Wallstrasse bridge in Ulm. K. SCHAECHTERLE. *Beton u. Eisen* (Germany), Jan. 5, 1931, V. 30, Heft 1, p. 14-18.—The Wallstrasse bridge (cf. *Bauzeitung* (Germany), 1906, Nos. 1, 7, 11) a three hinged open spandrel arch over the State Railroad built in 1905 has until recently given satisfactory service, with practically no maintenance cost. Clear span is 215 ft. with 187 ft. between the abutment hinges, and the rise 19 ft. The superstructure is 34 ft. wide. Arch concrete (stiff 1:3:5) showed a 57-day strength of 4800 lb. per sq. in. and the deck concrete (soupy 1:4¼) a 28-day strength of 2300 lb. per sq. in. In the last few years damp and wet spots appeared on the underside of the deck, water began to drip at the expansion joints and cracks and spalling became noticeable, particularly along the cantilevered side walk. Investigation revealed that slight changes in the original design would have prevented this deterioration. Where spalling had started, or where underspace of slab was wet, rusting of slab steel was found. In some places steel was exposed. Hair cracks and action of smoke led to much of rusting of reinforcement in portions above tracks. A lead and asphalt membrane used to water proof roadway, seems to have given fairly satisfactory service. Plain asphalt coating used for sidewalks proved inadequate due to cracking and to poor details along railing and curb. Damage was not serious enough to warrant expense of rewaterproofing the entire structure. Faulty joints along railing and curb were repaired and rusted steel was exposed and covered with fresh mortar. A groove was cut in the underside of sidewalk slab to act as a drip. Drains were provided along expansion joints. Paving was treated with a waterproof covering. After slabs were dry the entire soffit of deck slab received a waterproof coating. The following recommendations are made. Drainage should be promoted by covering upper surface of structure with a jointless asphalt coating or something similar. Both bearing surface and surface of structural slab should be given sufficient grade and lateral slope. Sliding plates over expansion joints should be avoided for ordinary spans. Percolated water should not be permitted to cross expansion joints. Waterproofing should not be given sharp bends and should be carried up a short distance at railing and curbs.—A. A. BRIELMAIER

BUILDINGS

Theatre and restaurant Des Ambassadeurs at Paris. L. GELLUSSEAU *Genie Civil*, Jan. 24, 1931, No. 4, p. 77-81.—Structure comprises a large dining room with stage and a theatre with 1000 seats. Work progressed rapidly due

to use of a skeleton of reinforced concrete. The structure is 85 by 207 ft. and is very low—21.5 ft.—to be in keeping with general character of the Champs Elysées. With exception of two steel trusses entire structure is of reinforced concrete. To reduce time of placing, only cement of high strength was used. Entire job was completed in less than three months, a record according to European standards. Live load used in the design was 495 lb. per sq. ft. Tests showed short-time placing of concrete in no way impaired its strength.—M. A. CORBIN

New high buildings in Duesseldorf (Germany). WILHELM PETRY. *Zement* (Germany), Jan. 22, 1931, V. 20, No. 4, p. 79-82.—(1) In the enlargement and reconstruction of tall reinforced concrete office building, basement structure was protected against ground water by insulation of interesting design and will stand a water head of 16 ft. Superstructure is of skeleton type consisting of one inner and one outer row of columns. Span of horizontal connecting beams is 23.8 ft. (2) Municipal bath house has 2 indoor swimming pools, 82 ft. by 36 ft. and 82 ft. by 41 ft. Both rest on separate reinforced concrete foundations to prevent harmful stresses in case building settles.—A. E. BEITLICH

The new structures of the Bavarian Dairy Co. in Nuernberg (Germany). HERMANN MEIER. *Zement* (Germany), March 19, 26, 1931, V. 20, No. 12-13. p. 274-8, 304-6.—Remarkable features of a number of reinforced concrete dairy structures include ceilings with very large spans. Main building is reinforced concrete skeleton structure, 354 ft. long and 79 ft. wide. Distances between reinforced concrete columns are 19.7 to 37.5 ft. Main roof has reinforced concrete ceiling with spans from 78.7 to 121.4 ft. without supporting columns. Building is provided with 3 expansion joints. Reinforced concrete chimney was erected by Heine method with sliding forms. Two water tanks having a total capacity of 42,000 gallons and arranged circumferentially around chimney, are supported by a circle of 8 reinforced concrete columns, 65.6 ft. high, connected by 2 horizontal concrete rings. Foundation consists of 5 ft. reinforced concrete slab, an octagon with outer diameter of 41 ft. High early strength cement was used for chimney construction.—A. E. BEITLICH

DAMS

Construction features of Osage hydro-electric development. *Eng. News Record,* March 26, 1931, V. 106, No. 13, p. 523-528.—Many interesting construction features are involved in Osage hydro-electric project near Bagnall, Mo. Power house forms 511 ft. of total length of dam, which is 2543 ft. Spillways constitute 520 ft. of crest. The maximum height is 148 ft. Concrete was made at central mixing plant, so located that 70,000 cu. yd. was chuted directly into place. Remainder was hauled in hoppers on flat cars over a construction bridge from which it was placed either by chutes or gantry crane. Four 60-cu. ft. mixers were used, giving a maximum 24-hr. output of 5082 cu. yd., with a 2-min. mixing period. The total yardage was 551,000.—GLENN MURPHY

Placing 350,000-cubic yards of concrete in Ariel dam. W. A. SCOTT. *Concrete,* March, 1931, V. 38, No. 3, p. 13-16.—Equipment for recovery of sand, gravel and cobbles from river deposit, washing and screening, bulk cement storage, proportioning and mixing, and conveying by belt was installed in construction of Ariel dam and powerhouse. Aggregate was transported 1000 ft. from river and dumped into bunker. Bulk cement was hauled 12 miles to storage. Water-cement ratio in mixer batch ranged from 0.9 to 1.0, including aggregate water. Concrete batches were discharged from mixers into 2-yd. bottom-dump buckets, carried on flat-cars and pulled alongside any section of dam, where buckets were lifted by crane or derrick. Expansion and contraction joints for all sections were placed at average of about 30 ft. Joints in arch section were 2 ft. wide, allowing arch to be built up in 30 ft.

sections, separated by 2-ft. slots, to be filled with concrete after shrinkage. Spillway contraction joints were grouted through series of pipes after shrinkage. —C. BACHMANN

Ariel dam—example of modern engineering practice. *Eng. News Record*, March 12, 1931, V. 106, No. 11, p. 435.—A site considered so difficult that 26,000-ft. of drill holes were put down to explore it, a foundation excavated 125 ft. through gravel to bed rock on site of ancient waterfall, and careful concrete control are among construction features of Ariel Dam of Inland Power and Light Co., on the Lewis River in Washington, about 40 miles northwest of Portland, Ore. The 313-ft. structure, with 1300-ft. crest, of thin-arch type requires 300,000 cu. yd. of concrete, 95,000 cu. yd. of which are in the one gravity abutment. Sand and gravel are excavated, washed, and screened on the site. Aggregate is graded to two sizes, ¼-1¼ in. and 1¼-3. Mix is 1:3.4:6.6 by weight, with w/c of 0.97, giving average compressive strength of 2,600 lb. per sq. in. at 28 days. Three 2-yd. mixers, with time of mixing 2½ min., enable workmen to average over 1.6 cu. yd. per min. of placed concrete. Concrete is placed in 12 in. layers in 5-ft. lifts in gravity section and in 10 ft. lifts in arch section. Vibration of concrete is accomplished by a steel shoe carrying a small portable gasoline engine with an eccentric flywheel. To reduce heat generated and resultant shrinkage stresses, the arched section of the dam is built in sections 30 ft. long separated by 2-ft. spaces to be filled six months later. Studies with electric thermometers indicate arrangement is most successful. Tests indicate rapid cooling may be obtained by circulating water through pipes cast in the block. It was found that concrete placed at 60° F. will rise to a maximum of 140° F. during curing process.—GLENN MURPHY

MISCELLANEOUS

Concrete and reinforced concrete in the Colony. M. DAVID. *Summary Reports First I. C. for C. and R. C.*—This paper discusses general conditions under which concrete and reinforced concrete are built in the (Belgian) colony—reasons why works in durable material, especially concrete, have to be adopted; conditions to be fulfilled; remarks on colonial materials (cements, sand, gravel, and in particular, laterite concrete); means and methods of execution; labor; and costs.—LA TECHNIQUE DES TRAVAUX

The application of concrete in the Belgian Congo. R. COPPEE. *Summary Reports First I. C. for C. and R. C.*—Special conditions governing the choice of concrete as material for construction in the Belgian Congo are: Climatic effects on works, other destructive agents peculiar to tropical regions, delivery time and costs for other materials, poor quality of other materials.—LA TECHNIQUE DES TRAVAUX

The manufacture of pre-cast reinforced concrete pieces on the Belgian National Railway Co. C. LEMAIRE AND J. SCHOTTE. *Summary Reports First I. C. for C. and R. C.*—Organization of a work yard. Quality and specifications of raw materials. Description of the various operations involved in the manufacture. Descriptions of the articles manufactured and of the shuttering used in quantity production.—LA TECHNIQUE DES TRAVAUX

Experimental study and description of constructional elements in reinforced concrete. LOUIS BAES. *Summary Reports First I. C. for C. and R. C.*—(1) Farco metal as a material for centering and as reinforcement. Farco metal is formed of very thin steel sheets specially formed with projecting sides giving a certain stiffness. These sides are joined by a trellis with a mesh sufficiently small to retain the concrete. (2) Experimental study and description of reinforced concrete tubes made by rolling and intended to resist bending (process Tubarsi). The note describes the process of fabrication by rolling of tubes of concrete heavily reinforced longitudinally.—LA TECHNIQUE DES TRAVAUX

Special reinforced concrete piles. M. E. H. TJADEN. *Summary Reports First I. C. for C. and R. C.*—The "de Waal" system is as follows: hollow cylin-

drical pile is built of reinforced concrete and driven into ground down to a hard stratum; working through hollow an enlarged foot is formed; then the inside is filled with concrete. The piles may either be formed in situ or precast in pieces about 8 ft. long.—LA TECHNIQUE DES TRAVAUX

A casting operation of reinforced concrete piles. J. VILAR. *Summary Reports First I. C. for C. and R. C.*—Bridge required 800 reinforced concrete piles about 32 ft. 10 in. long, and as the space available for casting them was confined to 164 x 66 ft., a procedure had to be devised whereby they might be superimposed. This was effected by placing sheets of thick paper between adjacent piles to prevent bonding with one another, thus taking up the least possible space.—LA TECHNIQUE DES TRAVAUX

New methods of forming piles cast in situ. L. F. DE JESSEN. *Summary Reports First I. C. for C. and R. C.*—Foundation piles cast in situ are perfectly adapted to nature of soil and varied forces which a foundation must resist. Three solutions recently applied are: (1) Drawing of the tube with or without an interior stopper; (2) reinforced inclined piles cast in situ: (3) piles with a base cast in situ surmounted by a pre-cast shaft.—LA TECHNIQUE DES TRAVAUX

Investigation of the effect of frost on concrete with special consideration of the insulating effect of molds. ADALBERT POGANY. *Zement* (Germany), March 12, 1931, V. 20, No. 11, p. 246.—Tests were made with 3 in. concrete cubes. Molds consisted on 3 sides and bottom of 1-in. boards. Fourth side was exposed to temperatures of − 7.6° F. and could be provided with boards of ½ in. to 1¼ in. thickness. Thermometers were located at various places in concrete and temperatures were plotted. Different curves were obtained only during first 30 minutes; afterwards no differences could be noted with boards of different thickness. Exothermic process in concrete such that insulating properties of boards are of minor importance.—A. E. BEITLICH

Hastings promenade widening works. *Engineering* (England), April 3, 1931, V. 131, No. 3403, p. 446.—In rebuilding shore promenade at Hastings a sea wall was built at outer line by driving two rows of watertight sheet steel piling between which, after excavation, a concrete wall of "Sal Ferricite" was placed. This wall has a curved face and is finished off with a larger radius nosing to break force of the waves. Steel rails were used for reinforcement.—G. M. WILLIAMS

Pre-cast caisson impounding wall at Weston-Super-Mare. *Engineering* (England), April 10, 1931, V. 131, No. 3404, p. 477.—To increase bathing facilities at Weston-Super-Mare an artificial lake has been created on the beach to impound flood tide. A heavy wall was required to withstand strong gales together with high tides. Caissons of reinforced concrete 20 ft. long 12 ft. wide and 9 ft. high with the interiors divided into four equal portions by longitudinal and transverse wells were built with rapid hardening Ferrocrete cement in a sheltered cove and floated into position and sunk by filling compartments with sand. A 1:1½:3 concrete mix was used. Before floating caissons into position a base was prepared by excavating pits in shore sand and refilling with mass concrete and boulders; these pits being spaced so that each served to support adjoining ends of two caissons. Space of several feet between caisson ends was filled with mass concrete in which Stanton spun concrete sluice pipes were cast. After placing caissons the height was increased 2 ft. by addition of concrete blocks. A sloped retaining wall, rising at an angle of 30 deg. made up of boulders and faced with a thick layer of boulder concrete was placed on sea side of caissons with toe protected by sheet piling.—G. M. WILLIAMS

New ocean terminal at Halifax, N. S. *Canadian Engineer*, Feb. 10, 1931, V. 60, No. 6, p. 22.—Pier B unit recently constructed in Halifax Harbor consists of 30 reinforced concrete cribs, with depth of water varying from 35 to 45 ft. at low tide. Cribs are 108 ft. long, 52 ft. wide and 53 ft. 6 in. high for outer 18 and the same length but width of 42 ft. and depth of 42 ft. 6 in. for inner 12.

Special construction dock having a water depth of 35 ft. at low tide was built. Cribs were started on pontoons and after a certain height was built were launched and completed while floating. After completion these were towed to position and sunk by letting water into pockets. Cement Fondu, imported from England was used.—G. M. WILLIAMS

Cement calendar 1931. "Zement Kalender 1931." RIEPERT, 1930, Zementverlag, Charlottenburg, (Germany), RM. 3.60. Reviewed in *Bautechnik* (Germany), March 20, 1931, V. 9, No. 13, p. 198.—A few new additions to this book include special concreting methods, new application of concrete pump (Pumpkret-method). Chapters on cement pipes, concrete wells, cement roof tiles and concrete slabs for sidewalks are enlarged.—A. E. BEITLICH

Swimming pool at Chateau Laurier. *Canadian Engineer*, Feb. 29, 1931, V. 60, No. 8, p. 11.—Canadian National Railway has completed a modern swimming pool as part of therapeutic department at Hotel Chateau Laurier at Ottawa. Pool is 25 ft. wide, 60 ft. long and from 3½ to 9 ft. deep, lined with mosaic tile and lighted under water. Reinforced concrete was cast as a unit to avoid weakness at joints.—G. M. WILLIAMS

Physics for construction technique. "Bautechnische Physik." HIMMEL-STROH-MEYER, 1930, B. G. Teubner, Leipzig and Berlin (Germany), sixth edition, R. M. 5.40. Reviewed in *Zement* (Germany), March 5, 1931, V. 20, No. 10, p. 226.—This book treats physics from standpoint of construction practice and shows examples of physical reactions and of physical installations for construction work.—A. E. BEITLICH

Government grain elevator at Prescott. E. SMITH, *Canadian Engineer*, April 7, 1931, V. 60, No. 14, p. 19.—A 5,000,000-bushel terminal grain elevator, erected at Prescott, Ont., by Dominion Government, has length of 1285 ft. and width of 70 ft. made up of 125 circular tanks with an internal diameter of 20½ ft., and 190 interspace tanks. The height is 85 ft. and wall thickness 7 in. After driving piling and cutting them to grade an 18 in. thick reinforced concrete mattress was placed.—G. M. WILLIAMS

Concrete blocks supplant ties in experimental track. *Eng. News Record*, March 26, 1931, V. 106, No. 13, p. 514.—Track construction without ties, but with pairs of reinforced-concrete blocks connected by tiebars, is being tried on a practicable scale in Italy, and is to be applied more extensively. Blocks are 24x44 in. on the base, and 36x16 in. on top, top surface being sloped 1 in 20 to give the rail an inward cant. The thickness is 8 in. on outer side. Rail rests on a metal plate, under which is a ½-in. wood cushion block. Fastenings are provided so gage is adjustable.—GLENN MURPHY

Concrete units hold embankments. *Concrete Products*, March, 1931, V. 40, No. 3, p. 36-37.—Construction of an unusual retaining wall, involving use of approximately 48,000 5x8x12-in. concrete building tile furnished by San Angelo Building Material Co. was completed recently at plant of the West Texas Utilities Co., San Angelo, Texas. This retaining blanket is approximately 50x410 ft. and has a wall area of over 20,000 sq. ft. Panels, measuring about 10x12 ft. each laid on a 30 deg. slope, are tied together with reinforced concrete beams formed of same tile to prevent settlement of areas greater than 120 sq. ft. In this retaining wall, which protects a spray pond located near a small stream, concrete tile was bid on and placed for $1.35 per sq. yd. less than bid for rip-rap.—E. S. HANSON

Precast concrete pipe supports. *Conc. Building Conc. Prod.*, (England), April,1931, V. 6, No. 4, p. 1.—Reinforced concrete pipe trestles, consist of two raking columns, with two or more horizontal bracing members. Longest trestles are molded first, molds being cut short at level of top of one of braces, and a new end piece fixed. The concrete mix is 1:2:4, with rapid hardening cement. Trestles are from 7 ft. 6 in. to 20 ft. overall, and the heavier ones are lifted by shear legs after four or five days. They are slung into position from top and upper brace. Bottom of trestle is buried 1 ft. below ground level in a block of concrete. Another type of trestle consists of three units; footing, pre-

cast column of variable length and standard cap, precast or cast in situ Pyramidal footings are reinforced each way. A 9-in. recess is formed to receive column. A projecting tongue, specially reinforced, forms a key with reinforced concrete cap, which is afterwards grouted on.—JOHN E. ADAMS

Vibration—a new method of working concrete. M. BERENGIER. *Genie Civil* (France), Feb. 21, 1931, No. 8, p. 189-192.—Vibration of forms has given way to vibration of concrete mass itself, a process, which in France is designated as "pervibration." Apparatus generally consists of a shell, within which is enclosed vibrator proper, made of special steel, and operated with compressed air. This vibrator is immersed in the concrete, which acquires the properties of a liquid. When the vibrator is removed, no trace is found of its former location. In general, working of 1.3 cu. yd. may be effected with expenditure of 2 hp. hr. as recorded by compressor. A floating vibrator designed and used successfully on construction of concrete columns, is placed in the form and concrete poured. It rises with each successive pouring. Forms should be watertight. Advantage of vibrating process is elimination of uncertainties inherent in working concrete by hand. Tests made at the Laboratoire des Ponts et Chaussées demonstrate that high strengths may be obtained through vibration with poor mixes, strength, permeability and compactness tests yielding satisfactory results. Great possibilities for ornamental effects are afforded by vibrated concrete, in which the mortar coating of aggregate is very thin, and a compact mosaic effect is produced. Additional expenditures in using a vibrator are: purchase or renting of a vibrator; increased volume of aggregate, corresponding to increased compactness of concrete; watertight forms. Economy of this method results in: reduction of quantity of cement used through higher strengths obtained, which allow reduction of sections; elimination of all hand labor in working concrete; elimination of all finishing work due to very smooth surface.—M. A. CORBIN

Passenger quay "Stazione Marittima" in Triest (Italy). L. GSCHWEND. *Schweizerische Bauzeitung* (Switzerland), Jan. 24, 1931, V. 97, No. 4, p. 35-8.—New quay was connected with old structure by means of hinges to allow room for settling of old wall. Difficulties were encountered in driving reinforced concrete piling which supports 17 reinforced concrete piers of new structure. Piles were 17 in. in diameter and up to 72.2 ft. long. High grade steel tubing, 55.8 ft. long and 19.7 in. in diameter, with a smaller inner steel tubing (end of which was provided with pointed chrome-nickel steel head) were driven through 16 ft. layer of loose rocks. After reaching sound rock inner tubing was pulled and reinforced concrete pile made of high early strength cement was driven. Reinforcements consisted of 8 longitudinal rods and ¼ in. spiral windings. Molds were assembled on shore and concrete of piers was placed under water. Pozzuolana was added to mixture.—A. E. BEITLICH

Continuous conveying as an advancement in construction methods. E. RATHSMANN. *Beton u. Eisen* (Germany), March 20, Apr. 5, 1931, V. 30 Heft 6, 7, p. 117-121, 134-137.—A detailed review gives names of makers of typical machines and some figures on performances of continuous conveying methods for earth, aggregate, concrete etc. Belts and steel bands, in trough form, are suited for moving aggregates and concrete along a 16 to 20 deg. slope with a speed of 4 to 6.5 ft. per second. Troughs and pipes are employed in forwarding concrete, and vibrating troughs for moving dry materials. Conveyors for any slope include bucket chain excavators pneumatic conveyors and pumps. Fuller-Kinyon and Torkret machines are pneumatic conveyors. The former used chiefly for moving cement, depends on intimate mixing of material with air stream and latter is used for moving by means of air pressure, dry mixed concrete materials. The use of compressed air to convey fine concrete, mortar and grout, in a wet state, for cement gun work is widespread. Torkret system of pneumatic transportation of concrete calls for a dry mix, which is dropped into a pressure chamber and forwarded by compressed air through pipes (4 to 6 in. diam.) to discharge chamber where water is added and high velocity of dry

mix is vitiated by impact. Pumps for forwarding materials without the use of water as a vehicle present the latest and simplest method for granular and powdery materials with a small water content. Concrete with aggregate up to 1 in. can be pumped 330 ft. horizontally and 135 ft. vertically. This method almost revolutionizes concrete construction practice and often eliminates lifts, towers and belts.—A. A. BRIELMAIER

Details of movable forms and their operation. RICHARD T. MCKAY. *Concrete*, April, 1931, V. 38, No. 4, p. 12-14.—Forms for 8 concrete tanks with inside diameter of 29 ft. 9 in., walls 7 in. thick and 100 ft. high, have top diameter slightly larger and bottom diameter smaller. Mean thickness of wall 2 ft. from top is 7 in. Top of upper ring of segments is kept down 5⅝ in. so 2x6 in. joists space 16 in. center to center for supporting 1 in. decking can rest directly on them. Three rings of 2 by 8 in. segments, both top and bottom, are spiked together and laid so that, in their lengths, they lap each other. Staves are nailed to segments. I-beams over each row of tanks are set in moving forms and carried up with them to support roof slab over completed tank walls and also to support 2 by 6 in. joists that carry 1 in. decking placed over entire inside area of tanks. Deck is used for storage of bars placed as work proceeds, and is concreting deck. Exact placing is not necessary of jack rods. To pull between 60 and 70 sq. ft. of forms to each jack, spacing of 8 ft. is suggested. With forms 4 ft. deep on each side of wall, this spacing would give 64 sq. ft. of contact area to be pulled by each jack.—C. BACHMANN

Concrete mixture design controlled by Chicago & North Western. O. F. DALSTROM. *Concrete*, April, 1931, V. 38, No. 4, p. 21-24.—Instructions for designing concrete mixtures issued for information of its contractors and for use of its engineering and inspection forces by Chicago and North Western Railway are as follows: Form BD-1 is to be used whenever concrete is made. One copy completely filled out is to be sent to engineer of bridges as soon as concrete work is started, or before. The source of the aggregates should be determined and recorded on form. Soundness and colorimetric tests are recommended when value of aggregates is doubtful. Aggregates must satisfy specification requirements as to structure and cleanness. Concrete is noted on plans as Class A, B, or C. For Class A, strength at 28 days should be 3,000 lbs. per sq. in., requiring a maximum of 5½ gals. of water per bag of cement; Class B, 2500 lbs., with 6¼ gals. of water; Class C, 2000 lbs., with 7 gals. of water. Representative sample of aggregate is to be taken about a foot in from surface and half-way up stock-pile. Grading is next determined and entered, followed by field mix by volume and net quantity of mixing water. Yield is determined without calculations by reading scale. After mix design is complete, check should be made by trial batch, workability of which is measured by slump method. After concrete operations are under way, frequent determination of the moisture content of aggregates should be made and adjustment made at the mixer.—C. BACHMANN

Chicago and North Western instructs inspectors of concrete. O. F. DALSTROM. *Concrete*, March, 1931, V. 38, No. 3, p. 30-32.—At inauguration of controlling quality of concrete in field, railroad required own engineering and inspection forces to attend Portland Cement Association course of instruction on subject. Course has been repeated every spring since. Written instructions to inspectors form next step. Included are emphasis on w/c ratio method, list of specifications, directions, references and blank forms inspector should carry, list of equipment contractors and railway construction crews should have on job. Specific duties of inspectors are outlined in ten paragraphs.—C. BACHMANN

New quay walls in Hamburg (Germany), discussed from constructive and economic view point. K. BARITSCH. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 430-67.—A great number of new harbor structures are described in all details, with drawings, graphs, and static calculations, and costs of similar structures are compared. Following rein-

forced concrete structures are included: (1) Quay wall for Deutsche Werft, A. G., consists of 2 sections which are 820 and 666 ft. long. Latter part supports 80-ton traveling crane which reaches 165 ft. (2) Similar structure was built for wharf of Blohm and Voss. (3) Quay for Rhenania-Ossag Oil Co., shows interesting design (cf. *Zement* (Germany), Jan. 8, 1931, V. 20, No. 2, p. 39-40). (4) Reinforced concrete quay wall on east side of harbor in Cuxhaven (Germany). Detailed cost calculations are given.—A. E. BEITLICH

Dustproof concrete floors. *Conc. Bldg. Conc. Prod.* (England), March 1931, V. 6, No. 3, p. 42.—Concrete floors when properly prepared and laid should stand heaviest traffic without serious trouble from dusting or disintegration. Main causes of a dusting surface are: foreign matter in the aggregates; too much cement or fine aggregate; too wet or too dry mix; excessive trowelling; exposure of surface to action of frost; too rapid drying of surface; practice of sprinkling surface with neat cement. The finished surface should always be kept damp for 7 days at least, and increased resistance to wear may be obtained by treating surface with solution of sodium silicate. A 30.5 per cent solution of neutral sodium silicate diluted with about 4 times its volume of water, well brushed over floor and application repeated after 24 hr. has given good results. A vegetable oil such as tung oil may be applied neat, or diluted with turpentine or white spirit and freely applied, excess being wiped off after 2 hr. A second application is sometimes necessary if floor is very porous.—JOHN E. ADAMS

The eight-mile Cascade tunnel of the Great Northern Railway. A symposium. D. J. KERR, FREDERICK MEARS AND J. C. BAXTER. *Proc. Am. Soc. C. E.*, Feb., 1931, V. 57, No. 2, p. 184-268.—Symposium constitutes final professional record of this project and consists of Part 1, an historical outline; Part 2, a description of the surveying methods; and Part 3, detailing methods of construction. This 7.79 mile tunnel (longest railroad tunnel in the Western Hemisphere and fifth longest in the world) replaces a 2.7 mile tunnel completed in December, 1900, and which lowered the Cascade crossing from 4,059 to 3,383 ft. elevation and replaced temporary 4 per cent switchback grades over pass to 2.2 per cent. Tunnel is single track on a 1.565 per cent grade down to the west, concrete lined throughout and was completed in 3 years. The excavated section varied from 18 ft. wide by 25 ft. high to 22 ft. wide by 27 ft. 6 in. high. The tunnel is 16 ft. by 22 ft. 10 in. inside the concrete lining throughout. Excavation proceeded through a pioneer bore supplemented by a 622-ft. shaft in Mill Valley. This shaft cut the tunnel into lengths of 2.41 and 5.38 miles. Center heading followed by top heading and bench excavation was used. None of granitic tunnel muck was suitable for use in concrete lining and entire concreting operation was carried on via east and west portals. On account of conduits side walls were poured in two lifts. Concrete was mixed near the points of deposition. Sand and gravel bunkers equipped with batching devices were constructed near portals. Proportions were nominally 1:2:4, varied to meet variations in the aggregate. Diatomaceons earth was used. A special Blaw-Knox traveling steel form permitted the lining of 37.5 ft. of tunnel at each move. Travelers for form also carried a 14-cu. ft. concrete mixer and air placer and various chutes and elevators. The arch ring was placed pneumatically (cf. H. J. King in Appendix B of paper, p. 261-268). There were 8 of these form placers, 3 of which were operated from west portal and 5 from the east portal. As work progressed removal of forms was shortened from 48 hr. to as low as 8 hr. after final placement, without sign of injury to concrete. The highest rock temperature encountered was 75 deg. (115 deg. had been estimated) and concrete reached 93 deg. about 2 days after placement, cooling gradually to rock temperature after about 2 months. Total yardage of concrete lining was about 275,000.—H. J. GILKEY

Vibrated concrete coating protects underground pipe lines. *Concrete*, March, 1931, V. 38, No. 3, p. 23-24.—Water-cement ratio of 4¾ gal. to sack of cement was disclosed as proper mixture for vibrated concrete for coating underground pipe in studies in Portland Cement Association laboratory. Mixture of 1 part portland cement, 1½ parts well graded sand (0 to ¼ in.) and 3 parts of gravel (¼ to ⅜ in.) was made with 4¾ gal. of water per sack of cement. Mixture appeared dry and unworkable, but was readily placed in forms and around pipe with the use of electrically operated vibrator. Mixture was placed on 10 ft. section of 8⅝-in. O. D. pipe, making concrete covering 1¾ in. thick. Two piece form was made of No. 14 gauge galvanized sheet metal. American Steel and Wire Co., No. 12 welded wire fabric reinforcement, 2 by 6 in. was placed around pipe and held half way between pipe and form. Vibrator was fitted to top of form and operated as concrete was inserted. No honeycombing or segregation of materials resulted. Forms were removed in 24 hr. and concrete cured under damp burlap for 14 days.—C. BACHMANN

Public baths, Westminster. *Ferrocrete Snowcrete Bul.* (England), March, 1931, No. 22, p. 5.—Two swimming baths and over 100 slipper baths, offices, laundry, etc., are included in new reinforced concrete structure at Westminster. Floor of larger bath, which is 100 ft. by 35 ft. and 10 ft. deep at one end, is supported on longitudinal ferro concrete beams, resting on sleeper walls, carried down to firm foundations, and at 16 ft. centres. Additional support is given by columns. Side walls are finished by platform or footway, and floor, which is first covered with asphalt, and walls are lined with Sicilian marble. Staging for spectators is in ferroconcrete and the dressing boxes at gallery level can be folded back to form a dado, and give extra floor space. Smaller bath, which is 70 ft. by 30 ft. is similar except that it is lined with tiles and nosing and scum trough are white glazed fireclay. Roof is semi-circular barrel arch, carried by eight arch ribs springing from foundation slabs below basement level. Clear span is 68 ft. Tubular steel scaffolding was employed to support the timber centering for rib and concrete was mixed at street level and raised in skips by hoist crane at crown of arch. Mix for structural members was 11 cwt. Ferrocrete, 13½ cu. ft. sand and 27 cu. ft. ¾-in. Thames ballast. The mix for walls had only 6 cwt. cement.—JOHN E. ADAMS

Dome built of glass tiles. CRAEMER. *Beton u. Eisen* (Germany), Feb. 20, 1931, V. 30, Heft 4, p. 72.—In the construction of the Indo China Pavilion of French Colonial Exposition several transparent roofs were used. One of domes was built of square glass tiles 15 in. on edge by 3¼ in. thick. Tiles are separated by narrow reinforced concrete ribs, but are so shaped that the dome presents an all glass undersurface and concrete ribs are not noticeable. The dome has the form of a paraboloid of revolution with a diameter of 39.8 ft. and a rise of 4.1 ft. Necessary variation in width of ribs was too small to be evident. Horizontal thrust of dome was taken by a reinforced concrete ring, supporting members being subjected only to the vertical reaction of the dome. Forms consisted of frames with parabolic upper chords spaced at 15 in. Glass tiles were first placed on these open forms and reinforcement and concrete were then placed in spaces between tiles. Parabolic frames were used several times. Several slabs without curvature, were built with same type of glass tiles. These slabs spanned openings as large as 14 ft. 9 in. by 14 ft. 9 in. with a live load of 102 lb. per sq. ft.—A. A. BRIELMAIER

Precast concrete in sea defense work. *Conc. Bldg. Conc. Prod.* (England), Feb., 1931, V. 6, No. 2, p. 21.—Precast concrete units are being used in construction of new sea defense wall and promenade at Bridlington. A new 40-ft. promenade is carried between mass concrete sea wall 1500 ft. long and 16 ft. high, faced with precast concrete units, and existing coast. Reinforced concrete shelters are formed on the inner side of new promenade, with 8 or 9-in. reinforced concrete roof slab carried on 12 by 9-in. beams at 12 ft. 6 in. centres. Beams are supported on back wall and precast columns on front. Columns

are 11 by 11 in. for the lower third of their height and taper to 9 by 9 in. at top. Bases are 15 by 15 in. high, and ornamentally reduced to section of base of column. Similarly ornamented caps are cast on columns. Pine molds were made on the site by the contractors and handled by loco. crane. Reinforcement projects well out of the cap for bonding into beams. Wall facing blocks are composed of 1:2:4 mix, aggregate being obtained from the beach. The seaward faces have a 9 in. facing of concrete made with crushed granite chippings. Blocks ranged from 6 ft. 6 in. by 2 ft. 6 in. by 1 ft. to 3 ft. 6 in. by 2 ft. 6 in. by 1 ft. The upper edges are beveled on the face, each course being stepped back 8 in. at the lower joints to 4½ in. at the top. Triangular recesses are provided in the sides for keying purposes. The promenade paving slabs are 3 ft. by 3 ft. 3½ in. by 4 in. thick, and are made alongside the site on timber molds capable of taking two slabs. The bottom 3 in. of slab is mixed in proportion of 6 parts beach gravel to 1 part portland cement. The 1-in. facing is made with 4 parts crushed granite and black sand to 1 part cement, to eliminate glare. Four recesses are cast on the lower edges of the slab to simplify lifting and transport, and sheets of mesh reinforcement are inserted to prevent cracks.—JOHN E. ADAMS

Screw piles and cylinders. *Magazine Tecnico* (Argentina), Sept.-Dec., 1930.—The screw pile and cylinder are not new but until recently they were impracticable because of complicated equipment required. Usual forms are (1) solid steel piles about one foot in diameter united in lengths of 15 ft. and used for docks, (2) screw cylinder made in diameters up to 4 ft. and lengths of 10 ft. Material is removed as cylinder is lowered and then cylinder is refilled with concrete. (3) Concrete pile with a screw shoe of steel, screw being driven by a steel shaft running through center of concrete. Exterior of the concrete shell is protected by light steel plates. This type is filled with concrete after the drive shaft is removed. Screw piles have screw with diameter about twice that of pile which gives them a greater bearing area. They can be put down and tested and then unscrewed if location is found unsatisfactory. The Braitwaite Co., Ltd., London, have adapted electric capstan to placing of screw piles and cylinders so that now they can be placed to depths of a 100 ft. with rapidity and economy. Electric capstan consists of a framework of steel on which electric motor and reduction gears are mounted. Cylinders are made of steel or cast iron and where diameter is large enough, internal unions are used. The section of the screw or helix is calculated considering screw as a curved cantilever under uniform load using the formula $f = \frac{3\,W\,(R+r)\,(R-r)^2}{\pi\,r\,t^2\,R^2}\left(1-\frac{(R+2r)}{(3R+r)}\right)$ where f = stress at point where helix joins the cylinder. W = load on cylinder. R = exterior radius of helix. t = thickness of the helix at point where it joins the cylinder. The pitch of the helix depends on (a) the material on which the helix is to work, (b) the rate of penetration necessary, (c) the power available in the capstan. The best efficiency is obtained at a velocity of between ½ and 1 revolution per minute of the screw. The power required for the capstans is between 5 h. p. for the small piles and 30 h. p. for the cylinders 10 ft. in diameter. Cost is much lower than for foundations or caissons placed by other methods.—C. G. CLAIR AND M. N. CLAIR

ROADS AND PAVEMENTS

Field determination of cement content of paving concrete. H. ALLEN. *Eng. News-Record*, March 26, 1931, V. 106, No. 13, p. 513.—For determining cement content of mixes proportioned by weight, field equipment consisting of a cast-aluminum measure with a capacity of ½-cu. ft., platform scales, a rod, and a trowel has proven satisfactory. When a standard method of tamping and weighing sample is followed, actual amount of cement in a given sample can be determined quite accurately.—GLENN MURPHY

Concrete roads in Belluno (Italy). *L'Industrie Italiana del Cemento* (Italy), 1930, No. 10; reviewed in *Ostr. Bauzeitung* (Austria), Feb. 14, 1931, V. 7, No. 7, p. 84-5.—Extreme care and construction control is absolutely necessary for good results. Eight sections of highway were built with concrete of varying composition. Road foundation consisted partly of old underground, partly of 6 in. concrete layer. Wire netting was imbedded in concrete. Mixtures contained 440 to 770 lb. cement per 35.31 cu. ft. Reinforcements were prepared by patented special method. Street will be subject to inspections from time to time.—A. E. BEITLICH

Bituminous surfaced concrete sidewalks. A. W. HADDOW. *Canadian Engineer*, March 31, 1931, V. 60, No. 13, p. 19.—City of Edmonton, Alberta, has developed a bituminous surfaced concrete walk consisting of a 4-in. base of 2500-lb. concrete topped with a 1-in. sheet asphalt mixture. Concrete is placed in the usual manner after grading and placing forms with side and transverse retaining shoulders built up to finished surface. Asphalt surface is rolled in place between shoulders several weeks later. Cost is 30c per sq. ft., including all charges.—G. M. WILLIAMS

WATERWORKS

Filtration plant at Brantford, Ont. F. P. ADAMS, *Canadian Engineer*, Feb. 17, 1931, V. 60, No. 7, p. 15.—Brantford, Ont., has completed construction of new pumping and filtration plant, the latter consisting of six 100,000 gallon gravity type rapid sand filters together with mixing chamber, sedimentation basins and filtered water storage reservoir. To insure watertightness a concrete mix of 1:1½:3 was used. The roof of pumphouse and filter building was made of Haydite concrete.—G. M. WILLIAMS

Filters and pumping station connected by concrete conduit. WM. C. MABEE. *Eng. News Record*, April 23, 1931, V. 106, No. 17, pp. 690-692.—Following completion of second large conduit connecting White River filters of Indianapolis Water Co. to Riverside pumping station, careful leakage tests indicated an extremely tight line. Results were more interesting in that conduit passes under Fall Creek and was laid in water-bearing sand and gravel, dewatered with well points. Line is of precast reinforced concrete, consisting of 4400 ft. of 60-in., and 700 ft. of 54-in. copper-bell pipe, in 12½-ft. lengths, 6 in. thick. Pipe was cast in steel forms, with flanged sheet-copper cylinder in pipe bell to provide for expansion and to take up inequalities of grade. The circumferential reinforcement was designed for a head of 25 ft., and longitudinal reinforcement comprised 0.22 per cent of shell area. The concrete was 1:1½:2½, and pipes were moist cured.—GLENN MURPHY

Water purification plant at Ottawa, Ont. A. D. STALKER, *Canadian Engineer*, Feb. 10, 1931, V. 60, No. 6, p. 15.—Work was started in 1929 on a water purification plant having an initial capacity of 35 million gallons daily. Plant, covering an area of 384 by 342 ft. consists of mixing tanks, settling basins and filters together with a filtered water reservoir of concrete. Mix having a slump of 3 to 4 in. with 4.9 to 5.1 gal. of mixing water per bag of cement resulted in strength of 3000 lb. per sq. in. at 28 days. Concrete walls were cured by sprinkling from a ½ in. pipe, perforated at 1 ft. intervals, placed above wall. Flat surfaces were cured by covering with sawdust kept moist by sprinkling. All concrete was kept wet for 14 days.—G. M. WILLIAMS

The use of separated sizes of coarse aggregate in the proportioning of concrete for highway construction. R. W. CRUM. *Nat. Sand Gravel Bull.*, Feb., 1931, V. 12, No. 2, p. 48-52.—Use of coarse aggregate separated into two or more sizes in construction of concrete pavements has been found to be feasible. It does not unduly complicate operations and added cost is small.

Two things can be accomplished by shipping coarse aggregate in two or more sizes and combining them at batching plant. (1) A better grading, having lower void percentage, may be secured than would be possible by taking run of pit. (2) Evil of segregation of sizes in stock piles and cars can be minimized, thus assuring delivery of more uniformly graded aggregate to individual batches. Whether or not this will result in more uniform concrete depends upon importance of coarse aggregate grading in particular concrete mixture being used. From a study of costs of materials and handling, effect of batch size upon cost, workability, durability, and strength and uniformity, it appears that separated sizes of coarse aggregate can be used with uniformity in results if coarse aggregates are furnished in several sizes: and that mixtures with an excess of mortar can be used to produce uniform results without dependence upon control of aggregate grading. An advantage of use of mixtures containing higher percentages of mortar is ease with which workability can be maintained without extra precautions. The controlling factor in choice of type of mixture under specific conditions is largely that of economy in composition of concrete.—P. McKIM

Silicate of soda street foundations. LUIGI BASCHIERI. *Le Strade*, Dec., 1930, V. 12, No. 12, p. 429-440.—Good results have been obtained, in comparison with macadam, with street foundations constructed with calcareous materials and silicate of soda in accordance with the rules given by Guelle of France based on his experiments in 1922. The mix consists of about 1 cu. yd. of graded calcareous stone up to 2 in. combined with ⅓ cu. yd. of calcareous sand or dust resulting from crushing stone and approximately 10 gal. of 35° Beaume sodium silicate. At present experiments are being made to investigate the effect of the variation in proportions on the characteristics of this sodium silicate concrete. A calcareous rock and a quartz rock were used in mixtures with sodium silicate solutions of various densities and compression specimens made from these mixtures were tested at several ages up to 60 days. Limestone aggregate was much superior to the quartz and mixtures with aggregate graded below ⅜ in. were better than those with excess of larger, or smaller aggregate.—C. G. CLAIR AND M. N. CLAIR

SHOP MANUFACTURE

Concrete garden frame. F. BURREN. *Conc. Build. Conc. Prod.* (England), April 1931, V. 6, No. 4, p. 76.—A design for a reinforced concrete garden frame is illustrated, and its advantages outlined. Vibration of concrete into position is recommended, and also a mix of 1:1½:3 with ⅜-in. crushed granite —JOHN E. ADAMS

Precast concrete factory at Liverpool. *Conc. Building Conc. Prod.* (England), April, 1931, V. 6, No. 4, p. 71.—Main features of products, and a description of the nature of recent contracts carried out by a well known firm of precast concrete manufacturers, is given, and illustrated.—JOHN E. ADAMS

"Eternit-Durasbest" and its application in the building industry. H. GROSSMAN. *Zement* (Germany), March 5, 1931, V. 20, No. 10, p. 221-4.—A new type of asbestos cement is used in manufacture of slabs and pipes. Eternit-Durasbest is made of a mixture of 1 part of asbestos and 6 to 7 parts of portland cement. Slabs are made 4 by 8 ft. Excess water is removed under hydraulic press. Pipes can be manufactured in various sizes and thickness of walls is made according to desired pressures.—A. E. BEITLICH

The modern way of manufacture of asbestos-cement pipes. E. LECHNER. *Zement* (Germany), March 12, 1931, V. 20, No. 11, p. 247-50.—A

comprehensive review of publications and patent literature on manufacture of asbestos-cement pipes. Detailed description of methods and installations are given and advantages and disadvantages are described. Methods are also judged from economic viewpoint. Special consideration is given to asbestos-cement pressure pipes, manufacturing methods and their application. Spinning of asbestos pipes resulted in very porous product not suitable for high pressures. Great differences in specific gravities of cement and asbestos cause considerable separation of constituents during spinning process.—A. E. BEITLICH

Profitable manufacture of concrete building units. FRED A. SAGER. *Concrete*, April, 1931, V. 38, No. 4, p. 17-20.—Several methods of handling raw materials are given to illustrate analysis of cost of this part of plant operation. Three methods include placing raw materials in storage by hand, by elevator with bins on ground, and with full elevated bins. In case taken as example, handling operations from car to mixer are (A) shoveling from car to pile, and (B), shoveling and wheeling in wheelbarrows or wheel cart from storage to mixer. Labor rate of 75c per hour, gives cost of 12.5 cents per 1,000 lb. for putting cinders into storage piles. For handling sand or stone as aggregate, suitable labor performance, applicable to those materials, should be used. For wheeling from storage pile to mixer, with 2¼ cu. ft. wheelbarrows, an average run of 50 ft. is taken. Time for filling one wheelbarrow is 40 sec., wheeling, dumping and returning 60 sec., or at a rate of 36 trips per hour, which moves 81 cu. ft., or 4,500 lb. per hour, at a cost of 16.66 cents per 1,000 lbs. Proper runways are effective in reducing cost of operations.—C. BACHMANN

Modern manufacture and use of concrete products and artificial stone. VON MENG. *Beton u. Eisen* (Germany), Oct. 20, 1930, V. 29, Heft 20, p. 357-362.—Modern concrete products have attained their present importance because of the simplicity of manufacture and freedom possible in selection of shape, size and adaptability. Methods of mix design employed in heavy concrete construction cannot be applied to concrete products without some changes. Very thorough study of materials and proportions is essential before production is begun. Fundamental theory, such as that concerning the effect of quantity of water, is same as for mass concrete. In addition to the usual sand gravel and stone, and the natural light weight stones, pumice, lava and tuff there are artificial aggregates including slag, cinders, artificial pumice, porcelain grits, cork, asbestos, treated sawdust, wood wool and others. Selection of aggregate is governed by properties desired in the product; flexibility, compressive, tensile or bending strength, fire resistiveness, impermeability, insulation against heat, sound and electricity, etc. In many cases these properties may be further increased by addition of substances having a chemical or physical effect, as for instance in manufacture of porous concrete. Type of cement and method of concreting have a bearing on resulting properties of the product. Concrete may be poured, vibrated, compressed, squirted or centrifugally cast. Outstanding development is centrifugally-cast pipe, (in Germany mainly Vianini, Dywidag and Hume pipes). These have been replacing to a large extent the welded and riveted iron pipes and wooden pipes. A recent addition is "asbestos cement pipe" (Asbestzenrentrohr) with 2 to 4 in. inside diameter, capable of withstanding 220 lb. per sq. in. pressure and a worthy competitor of iron, steel and cast iron pipes. It is cheaper than metal pipes, light in weight, has a high elasticity and is tough, is a non-conductor of electricity and is not affected by salts or acids within certain limits. Use of concrete for posts, trolley poles, street light poles, etc., has opened up a large field, especially for the centrifugally-cast pole. Concrete paving tiles are cast under 5150 lb. per sq. in. pressure, in pieces 14 by 14 by 3 in. and placed on a subpavement of 1:12 concrete 1¼ to 2 in. thick. Joints were filled with cement grout. Concrete blocks and artificial stone are well adapted to remodeling. —A. A. BRIELMAIER

New Oklahoma pipe plant. DUDLEY W. MOORE. *Concrete Products*, March, 1931, V. 40, No. 3, p. 22-24.—Plant of Oklahoma Cement Pipe Co., Tulsa, Okla. has a daily capacity of 60 to 70 tons of manufactured products of which storm sewer pipe is principal product. Sizes range from 12-in. to 60-in. diameter, made on a machine of revolving platform type. Octagon steel storage bin with five compartments for the storage of stone, sand and cement has capacity of 800 tons. At the top of bin is a turntable distributing device controlled by a wheel and bevel gears from the batching floor beneath the bin so that separate sizes of material are delivered to the various compartments. Discharge is by gravity through a gate into a weighing batcher. Company also manufactures transit mixed concrete.—E. S. HANSON

Betonwerkstein (cast stone). FRIESECKE. *Betonwerk* (Germany), Feb. 28, 1931, No. 9, V. 19, p. 113.—Subject is divided into eight parts: (1) Development of manufacture, selection and use of various aggregates, methods of fabrication and evolution of present designation; (2) cast stone is defended, it is not an imitation; (3) it is a suitable building material, it is strong, withstands weather and is beautiful; (4) present day knowledge of portland cement and concrete mixtures coupled with skill of fabricators is guarantee of good quality; (5) long list of meritorious installations from year 1869 to the present are listed in details; (6) law against willful disparagment of another's product is quoted; (7) observations of experiences of American cast stone manufacturers cited; (8) economy in use of cast stone. Artisans in technique of fabrication are being trained. No pains are being spared in overcoming prejudices and placing cast stone in high place it deserves.—H. FRAUENFELDER

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Institute Objects, Organization and Committees

The American Concrete Institute had its beginning in 1905 with organization (incorporated District of Columbia 1906) as the National Association of Cement Users. In 1913 its charter was amended with change of name. With a membership of about 2500 it maintains a continuous program in studying the properties of concrete and reinforced concrete and their constituent materials; considerations of concrete design, manipulation, application, service. Its aims are to increase the knowledge of concrete and to extend the use of that knowledge. It maintains no laboratory, employs no field force. The product of its work is in papers, committee reports and in discussions of them—correlating the experience of many agencies, public and private.

The Institute holds an annual convention as a forum for the presentation and discussion of outstanding developments in the theory and practice of its field, publishes the Journal of the American Concrete Institute monthly, except July and August. The Journal is in three sections: *News Letter*, for the news of the society and informal contributions tending to stimulate its technical program; *Proceedings* for technical papers, reports, discussions; *Abstracts*, consisting of digests of the current periodical literature of cement and concrete.

Proceedings have been published each year beginning 1905; in 1913, 1914, 1915 (Vols. 9, 10 and 11) these were issued periodically in Journal form. A return to the annual volume was made in 1916 with Vol. 12.* Completing Volume 25 in June 1929, Journal publication was again undertaken, No. 1 of Vol. 26 being issued in November, 1929.

The work of the organization is governed by the Board of Direction with 15 members (see By-Laws) and administered by the Secretary, appointed by the Board.

The Board divides the labors of formulating details of the creative program among three major committees: Advisory, originating technical committee assignments; Program, develop-

ing the annual convention; Publications, in general charge of the JOURNAL, which is edited by the Secretary. These committees report to the Board for approval of their undertakings.

In all of this the Institute constitutes a clearing house for the information of and for its chosen field. It is dependent on many public and private agencies and the work of individuals whose studies produce the data for which the Institute offers channels of dissemination and a forum for critical consideration and discussion.

Board of Direction and Executive Committee

The Board of Direction meets at the time of the annual convention; again in the spring (usually early May) and in the Fall (usually the first week of October). Between meetings its authority is vested in the executive committee consisting of the President, Secretary and three other Board members, named by the Board:

D. A. ABRAMS
E. D. BOYER
S. C. HOLLISTER
M. M. UPSON
HARVEY WHIPPLE

*Vol. 11, 1915 is incomplete, numbers 9 to 12 of that year never having been issued. Several numbers of the *Journal* for 1913, 1914 and 1915 are out or print as are also a few of the early volumes of *Proceedings*. Of the *Journals*, Nov. and Dec. 1913; Jan., Feb., Mar. 1914; and Jan., Feb., Mar., Apr., May, June, July and August 1915 are available and are for sale at 50 cents each. A limited supply of annual volumes, 2, 6, 12, 13, 14, 16, 17, 18, 19, 20, 21, 22, so far as available are for sale to non-members at $8.00 each; Vol. 23, 24 and 25 at $10.00 each. The member price for available volumes to and including Vol. 25, 1929, is $5.00 each.

The new JOURNAL is sent to all members (paid for by an allotted portion of the annual dues). Members and JOURNAL subscribers desiring a complete bound copy of Volume 27, containing the *Proceedings* and *Abstracts* sections from the monthly JOURNAL (September 1930 to June 1931), in *addition* to the JOURNAL periodically, may buy it at $6.00. To non-members the price is $12.50.

Orders received from members or subscribers before September 1, 1931 for Volume 28 (September 1931-June 1932) will be filled at a special price—(not to exceed $3.50)—not more than two to each member. The Institute quotes this special price on orders received before the first JOURNAL issue of the volume, because it then knows the actual number of orders to be filled and can provide enough extra copies of each month's JOURNAL to make up the required number for binding. **After September 1, the price will be $6.00 to members,** because such orders must be filled from a limited reserve stock.

Advisory Committee

The Advisory Committee has 13 members: a Chairman, Secretary, the chairman of each of the nine departments of the Committee organization; together with the chairmen of the Program and Publications Committees:

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Chairman

F. R. MCMILLAN
Chairman Department 100, Research

R. W. CRUM
Chairman Department 200, Materials

A. W. STEPHENS
Chairman Department 300, Engineering Design

REXFORD NEWCOMB
Chairman Department 400, Architectural Design

MILES N. CLAIR
Chairman Department 500, Specifications

R. B. YOUNG
Chairman Department 600, Field Construction

W. D. M. ALLAN
Chairman Department 700, Shop Manufacture

RAYMOND E. DAVIS
Chairman Department 800, Use Requirements

F. H. JACKSON
Chairman Department 900, Joint Efforts

P. H. BATES
Chairman Program Committee

W. A. SLATER
Chairman Publications Committee

For several years an advisory committee has simplified the work of the Board of Direction (which under the By-Laws creates all committees, assigns their tasks and names the chairmen) by presenting to the Board recommendations as to technical committee studies and personnel.

In February 1929 the Board adopted a new committee plan and in several important details a new procedure. It did away with "standing committees" in the usually accepted sense. In general it avoided assignments to "fields of interest." Committees are created for one year and their membership appointed for one year. Assignments, for the most part are to definite tasks.

The nine department representatives on the Advisory committee, are to serve as "eyes" in their respective fields of interest upon developments, both active and potential, which demand Institute attention. As members of the Advisory committee they contribute to the formulation of recommendations to the Board in reference to the Institute's technical committee program. One or more other representatives (co-chairmen) are also named for each department, and while these additional representatives are not members of the Advisory committee they are looked to for suggestions to the Advisory committee as to the technical committee work.

It is the responsibility of the Advisory committee, to recognize the elements of unsolved problems, to systematize their solution in recommendations as to definite committee tasks and to seek out the men best fitted to the various studies to be undertaken.

If committee changes in personnel or assignment or new committee tasks seem desirable the department representatives are expected to suggest these changes for consideration of the Advisory committee as a whole, all of which work is made the basis of recommendations to the Board.

Thus the Advisory Committee (whose chairman and secretary serve as the executive group between meetings) will recommend to the Board the organization of committees, their assignments and chairmen, and to the President their membership, and will coordinate and systematize the Institute's technical committee activity.

Committee departments do not imply definite boundaries of interest and responsibility. Obviously department interests overlap. The function of a department is important in its *viewpoint*. The Research department brings to bear a research viewpoint upon a matter which may also be of interest in every other department. The viewpoint of the department has its bearing on each definite committee task.

A committee department represents a division of the subject matter of the field *only* with respect to a *viewpoint*. Its assignment is exclusive only in viewpoint. Committees in turn are assigned to definite tasks in which their department viewpoint is reflected.

In general the new committee organization now functioning successfully for two years, comprehends the use of larger total personnel than heretofore; the study and systematization of a much larger list of subjects; small working units; more definite and limited assignments; much quicker action; more general discussion and the recognition of individuals as "authors" of reports.

The immediate ends sought are:

(1) An Institute with a much larger and usefully active personnel.

(2) A monthly JOURNAL of outstanding importance in its service to every kind of activity in the field of concrete.

(3) The summarization of existing information and adaptation to form for more general use—with an American Concrete Institute Manual of Practice as an objective.

(4) A more energetic pursuit of needed information to fill gaps in existing knowledge by encouraging tests and field studies.

A considerable number of committees are set up under a plan which frankly recognizes that in many instances much of a committee's work is actually being done by one man. Numerous small "working units" have been—many others will be formed. One man will be named "author." He may or may not be chairman of the committee or working unit. He will be assigned the task of writing a report. Other members will be named as "critics."

Critic members of committees have two functions: suggesting to the author revisions in the original draft and writing independent discussion of the subject under assignment. A report from the author type of committee represents the views of a committee majority when a majority are at once in agreement or by consent of the author in making revisions. Otherwise it is the report of an individual. Thus the Institute provides a hearing for strong independent

views and brings them into the general forum (whether convention or JOURNAL or both) rather than submerging such independent views in the compromise of a majority report.

Program Committee

The Program Committee's work is to select from among papers and reports submitted to the Institute such contributions as will make, in the judgment of the committee, either a well rounded program or one which deliberately stresses definite subjects of interest, for the annual convention. The acceptance of contributions by the Publications committee is entirely independent of the convention and in no way commits the Program committee as to their use on the convention program. It is anticipated that the JOURNAL will publish papers, reports and discussion considerably in excess of the requirements of conventions. The Program committee's selection of a contribution for the program in no way implies that it is of greater value than others not so selected. Some contributions are better read by members from the printed record; often highly technical contributions of great value are poorly adapted to oral presentation and impromptu discussion. When the Program committee selects material the Publications committee will defer to its preference in publication before or after the convention presentation. The Program committee:

P. H. BATES
Chairman

HARVEY WHIPPLE
Secretary

S. C. HOLLISTER

A. E. LINDAU

ARTHUR R. LORD
Chairman Advisory Committee

W. A. SLATER
Chairman Publications Committee

Publications Committee

The Publications committee has the responsibility of the technical publications of the Institute and invites original papers furthering the objects of the society and discussions of published papers and reports. It passes upon the papers and the reports of committees submitted for publication. All contributions are received through the Secretary who is the editor of the JOURNAL and Secretary of the committee. The committee will make all reasonable effort to advise authors promptly as to availability of contributions for publication—subject to consideration by appointed critics to whom manuscript copies are submitted for their advice to the Publications committee. The Program committee is deferred to in all matters which that committee selects for the annual convention program. The Publications committee:

W. A. SLATER
Chairman

HARVEY WHIPPLE
Secretary

J. C. PEARSON

A. N. TALBOT

BENJAMIN WILK

ARTHUR R. LORD
Chairman Advisory Committee

P. H. BATES
Chairman Program Committee

Abstracts

The maintenance of the JOURNAL section of Abstracts (presenting digests of periodical literature of cement and concrete) is a part of the responsibility of the Publications committee involving a special labor of considerable detail which has been reassigned as the special task of the Committee on Abstracts, each of whose members is responsible for a division of the field. More than 100 technical publications are regularly scanned by a corps of abstractors. These contributions are edited by Dwight F. Jennings, with the advice and assistance of the committee:

J. C. WITT
Chairman (Materials)
H. F. GONNERMAN
(Properties of Concrete)
F. E. RICHART
(Engineering Design)
REXFORD NEWCOMB
(Architectural Design)
THEODORE CRANE
(Field Construction)
W. D. M. ALLAN
(Shop Manufacture)

Technical Committees

The technical committee organization as of July 1, 1931, is as follows:

DEPARTMENT 100—RESEARCH

To comprise all committee work in which the principal interest is in the interpretation of test data or the initiation or direction of tests.

Chairman—F. R. McMillan
Co-Chairmen—H. J. Gilkey and F. E. Richart

Committee 102—Volume Changes in Concrete

Author-Chairman—G. E. Troxell
Raymond E. Davis, H. J. Gilkey, Charles C. More

Committee 103—Effect of Curing Conditions

Author-Chairman—A. A. Anderson

Committee 104—Permeability of Concrete

Author-Chairman—L. G. Lenhardt

Ira L. Collier
Charles H. Scholer
R. H. Schroeder
M. O. Withey

Committee 105—Reinforced Concrete Column Investigation

Chairman—W. A. Slater

P. H. Bates
R. L. Bertin
F. R. McMillan
F. E. Richart
W. S. Thomson
W. F. Zabriskie

Committee 106—Effect of Vibration on Properties of Concrete

Author-Chairman—R. F. Leftwich

Committee 107—Properties of Job-Cured Concrete at Early Ages

Author-Chairman—Harlan H. Edwards

R. W. Crum
H. E. Davis
T. E. Stanton
M. A. Timlin

Committee 108—Properties of Mass Concrete

Chairman—Raymond E. Davis

P. H. Bates
R. W. Carlson
J. P. Crowdon
E. C. Eaton
F. W. Hanna
W. C. Hanna
G. W. Hawley
D.C.Henny
Louis C. Hill
W. L. Huber
G. W. Hutchinson
S. E. Hutton
H. L. Jacques
F. R. McMillan
S. B. Morris
W. A. Perkins
J. L. Savage
B. W. Steele
G. E. Troxell
A. J. Wiley
C. P. Williams
Hubert Woods

Committee 109—Plastic Flow

Author-Chairman—Raymond E. Davis

Hardy Cross
J. R. Shank

Committee 110—Use of Admixtures in Concrete

Author-Chairman—Ben Moreell
Vice-Chairman—Rex L. Brown

Committee 111—Effect on Concrete of Repeated and Reversed Loads

Author-Chairman—H. H. Scofield

Committee 112—Resistance of Concrete to Impact

Author-Chairman—W. M. Dunagan

Committee 113—Bond Stress

Chairman—F. E. Richart
Secretary—J. R. Shank

H. J. Gilkey
H. F. Gonnerman
T. G. MacCarthy
G. A. Maney
W. A. Slater
Albert Smith

Committee 114—Relations of Cement Content, Strength, Water-Ratio and Durability

Author-Chairman—L. W. Walter

F. R. McMillan
R. B. Young

DEPARTMENT 200—MATERIALS

To comprise all committee work in which the principal interest is that of the production and distribution of materials used in concrete work.

Chairman—R. W. Crum
Co-Chairmen—E. O. Sweetser and R. W. Johnson

Committee 201—Aggregate Specifications

Chairman—H. F. Clemmer

R. L. Bertin
A. T. Goldbeck
Fred Hubbard
F. H. Jackson
Herbert F. Kriege
H. J. Love
H. S. Mattimore
N. C. Rockwood
Stanton Walker
L. E. Williams

Committee 203—Cinders as Aggregate for Concrete

Author-Chairman—Einar Christensen

W. D. M. Allan
R. D. Bradbury
Miles N. Clair
Emil Praeger
George A. Strehan

Committee 204—Burned Clay or Shale Aggregates

Author-Chairman—George E. McIntyre

E. W. Dienhart
Iver Erickson
V. P. Jensen
R. F. Leftwich
A. B. Shenk
W. P. Stanton

Committee 205—Grading Aggregates for Handling on the Job

Author-Chairman—I. E. Burks

G. D. Durham
Alexander Foster
A. E. Stoddard

Committee 206—Welded Wire Fabric as a Reinforcing Material

Author-Chairman—R. D. Bradbury

DEPARTMENT 300—ENGINEERING DESIGN

To comprise all committee work in which the principal interest is that of the structural design or of the supervision of construction of such work.

Chairman—A. W. Stephens
Co-Chairmen—Hale Sutherland and H. N. Howe

Committee 304—Fire Resistance Standards in Design

Author-Chairman—N. D. Mitchell

R. L. Bertin
R. W. Johnson
F. R. McMillan
T. D. Mylrea

Committee 305—Structural Loads and Reductions in Design

Author-Chairman—R. L. Bertin

Peter L. Hein
H. N. Howe
George Thompson

Committee 307—Deflections of Reinforced Concrete Members

Author-Chairman—T. D. Mylrea

P. J. Freeman
Fred Kubitz
A. H. Saurbrey

Committee 308—Basis of Design for Hurricane Exposures

Author-Chairman—Albert Smith

D. C. Coyle
Hardy Cross
W. J. Knight
G. A. Maney
L. E. Ritter
W. C. Spiker

Committee 309—Minimum Permissible Thickness of Slabs and Walls

Author-Chairman—D. E. Parsons

Committee 310—Effect of Settlement of Foundation in Reinforced Concrete Structures

Author-Chairman—Hardy Cross

Committee 311—Moment of Inertia of Reinforced Concrete Members

Author-Chairman—F. E. Richart

M. B. Lagaard
G. A. Maney
Hale Sutherland
L. C. Urquhart

Committee 312—Plain and Reinforced Concrete Arches

Author-Chairman—Charles S. Whitney

DEPARTMENT 400—ARCHITECTURAL DESIGN

To comprise all committee work in which the principal interest is that of the architectural design or of sculpture or of the supervision of such work.

Chairman—Rexford Newcomb
Co-Chairman—O. F. Johnson

Committee 404—Composite Concrete Structures

Author-Chairman—J. Morrow Oxley

Committee 405—Economics of Tall Building Design

Author-Chairman—R. R. Zipprodt

S. C. Hollister
A. W. Stephens

Committee 406—Use of Light Weight Concrete in Buildings

Author-Chairman—Frank A. Randall

Frank Brown
W. S. Wolfe

Committee 407—Painting on Concrete Surfaces

(undergoing reorganization)

Committee 408—Recommended Practice for Use of Color in Concrete

Author-Chairman—W. D. M. Allan

J. G. Ahlers
Joseph W. Ayers
C. A. Cooke
John Mott
Rexford Newcomb
M. R. Rust
Edward M. Waldo

DEPARTMENT 500—SPECIFICATIONS

To comprise all committee work in which the principal interest is that of the production of specifications for material, services or workmanship on behalf of the owner.

Chairman—Miles N. Clair
Co-Chairman—P. J. Freeman

Committee 501—Standard Building Code

Chairman—A. W. Stephens
Secretary—R. R. Zipprodt

R. L. Bertin
H. P. Bigler
R. B. Bradbury
Frank E. Brown
A. E. Lindau
Arthur R. Lord
F. R. McMillan
N. D. Mitchell

Miles N. Clair
Theodore Crane
Hardy Cross
S. C. Hollister
H. N. Howe
R. W. Johnson
T. D. Mylrea
F. E. Richart
W. A. Slater
Hale Sutherland
W. S. Thomson
L. C. Urquhart
R. B. Young

Committee 502—Mixing and Placing Concrete in Buildings

Author-Chairman—Arthur R. Lord
A. S. Douglass
A. B. MacMillan
J. Thumley
W. F. Way

Committee 503—Fabricating and Setting Reinforcing Steel

Author-Chairman—W. F. Zabriskie
R. L. Bertin
R. W. Johnson
A. R. Lord
W. S. MacKenzie
W. S. Thomson

Committee 504—Specifications for Ready-Mixed Concrete

Author-Chairman—Miles N. Clair
Alexander Foster, Jr.
P. J. Freeman
Frank I. Ginsberg
R. S. Greenman
W. E. Hart
C. E. Nichols
H. F. Thomson
Stanton Walker
R. B. Young

Committee 505—Reinforced Concrete Chimneys

Author-Chairman—E. A. Dockstader
William Jassoy
E. R. Maurer

Committee 506—Concrete Specifications for the Small Job

Author-Chairman—Arthur R. Lord
Theodore Crane
P. J. Freeman
S. C. Hollister
W. F. Way

507—Purchase Specifications for Metal Supports for Reinforcement

Author-Chairman—W. E. White

Committee 508—Concrete Foundations for Steel Buildings

Chairman—J. E. Hough

DEPARTMENT 600—FIELD CONSTRUCTION

To comprise all committee work in which the principal interest is that of monolithic construction, including job equipment, organization, etc.

Chairman—R. B. Young
Co-Chairmen—R. C. Johnson, Nelson L. Doe

Committee 601—Measuring Materials for Concrete

Author-Chairman—R. B. Young
I. E. Burks
A. A. Levison
H. C. Ross

Committee 602—Developing Latent Strength of Concrete

Author-Chairman—H. F. Gonnerman

Committee 603—Design and Operation of Central Mixing Plants

Author-Chairman—Frank I. Ginsberg
Joseph E. Burke
Miles N. Clair
N. D. Crowley
G. L. Lucas
A. W. Munsell
H. F. Thomson
Fred C. Wilcox

Committee 604—Winter Concreting Methods

Author-Chairman—R. C. Johnson
I. E. Burks
W. S. McNaughton
L. J. Street
R. B. Young
Nelson L. Doe

Committee 605—Settlement of Concrete In Forms

Author-Chairman—James Lindon

Committee 607—Use of Cement in Bulk

Author-Chairman—Herbert Coffman
Peter F. Stauffer
Malcolm H. Ulman
A. G. Watt
Fred C. Wilcox

Committee 608—Sliding Formwork

Author-Chairman—L. Boyd Mercer

John H. Heindel
W. R. Sproul
Edmund Wilkes, Jr.

DEPARTMENT 700—SHOP MANUFACTURE

To comprise all committee work in which the principal interest is that of the manufacture and distribution of factory made concrete products.

Chairman—W. D. M. Allan
Co-Chairmen—C. F. Buente, C. A. Wiepking

Committee 701—Standards for Concrete Masonry Units

Chairman—Benjamin Wilk
Secretary—R. E. Copeland

Nolan Browne
Einar Christensen
E. W. Dienhart
C. J. Herzog
D. E. Parsons
George O. Payne
A. G. Swanson

Committee 702—Prevention of Crazing

Author-Chairman—P. H. Bates

F. O. Anderegg
E. D. Boyer
John J. Earley
J. C. Pearson

Committee 704—Cast Stone Standards

Chairman—Louis A. Falco
Secretary—C. G. Walker

P. H. Bates
George J. Eyrick
Herman Fraunfelder
J. W. Lowell
Ernest Rackle
C. Van de Bogart

Committee 705—Weather Resistance of Concrete Masonry Units

Author-Chairman—C. A. Wiepking

W. D. M. Allan
Einar Christensen
Dan F. Servey

Committee 706—Concrete Pipe

Chairman—W. J. Schlick

Committee 708—Recommended Practice for the Manufacture of Concrete Masonry Units

Chairman—P. M. Woodworth

E. W. Dienhart
C. J. Herzog
M. I. McCarty
Eugene F. Olsen
W. P. Stanton
Stanton Walker

Committee 709—Specifications for Concrete Burial Vaults

Chairman—W. D. M. Allan

C. A. Berg
H. F. Hanneman
Carl Head
H. A. Ledyard
F. J. Mead
Harry Samson
J. A. Stuart
F. H. Van Ness
C. A. Wiepking

DEPARTMENT 800—USE REQUIREMENTS

To comprise all committee work in which the principal interest arises from the requirement of unusual or special qualities in concrete on account of its use in less common construction or where subject to extraordinary exposures.

Chairman—Raymond E. Davis
Co-Chairman—W. F. Way

Committee 801—Durability of Concrete

Chairman—A. E. Lindau
Secretary—F. R. McMillan

P. J. Freeman
G. F. Loughlin
E. Viens
L. W. Walter
R. B. Young

Committee 802—Portland Cement Floor Finish

Chairman—John G. Ahlers
W. E. Hart
Millard F. Bird

DEPARTMENT 900—JOINT EFFORTS

To comprise all committee work done by the Institute jointly with representatives from other societies and work in which the principal interest is clearly in two or more of the foregoing departments.

Chairman—F. H. Jackson
Co-Chairman—Cloyd M. Chapman

Committee 901—Concrete Culvert Pipe Standards

Chairman Institute Representation—B. S. Pease
F. F. Longley

Committee 902—Concrete Pavement Standards

Chairman—F. C. Lang

Committee 903—Joint Committee on Specifications for Concrete and Reinforced Concrete

Chairman Institute Representation—S. C. Hollister
J. G. Ahlers
Ben Moreell
F. H. Jackson
N. M. Stineman

Committee 904—A. C. I. Representation Sectional Committee on Plastering

J. C. Pearson

Committee 905—A. C. I. Representation Sectional Committee on Specifications for Portland Cement

J. G. Ahlers
E. D. Boyer
W. K. Hatt

Announcements of committee changes, of the completion of committees not now fully organized and of new committees will be made from time to time in the JOURNAL —*News Letter* section.

By-Laws

American Concrete Institute

ARTICLE I

Members

Sec. 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to cooperate in the purpose of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at the time, with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary and by the payment of the annual dues, and unless these dues are paid within 60 days thereafter, the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Sec. 6. The Board of Direction may confer honorary memberships in recognition of services of an extraordinarily meritorious character before the Institute. Honorary members shall be entitled to full membership privileges without the payment of dues.

ARTICLE II

Officers

Sec. 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction at least six months prior to the Annual Convention. The President, Vice-President, Secretary, Treasurer and the Directors from the six geographical districts shall be ineligible for membership on this committee.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next

Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 120 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 60 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 60 days before the Annual Convention to the members of the Institute for letter-ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

Sec. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability of or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

Sec. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

Sec. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

Sec. 7. The Board of Direction shall appoint a Committee on Resolutions to be announced by the President on the first regular session of the annual convention.

Sec. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

Sec. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

Sec. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

Sec. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

Sec. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III

Meetings

Sec. 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of Convention.

Sec. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

ARTICLE IV

Dues

Sec. 1. The fiscal year shall commence July 1st.

Sec. 2. The annual dues shall be twelve dollars and fifty cents ($12.50) payable annually in advance from the first of the month following notification of the applicant of his election by the Board of Direction.

Sec. 3. Each member shall be entitled to receive one copy of one volume of the Proceedings for each membership year and additional volumes at a price fixed by the Board of Direction.

Sec. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

ARTICLE V

Standards

Sec. 1. Proposed new or revised Standard Specifications, Standard Practice, and Standard Definitions when approved by a majority voting in the committee in which they originate, shall be submitted, in the form adopted in the Standard Form of Standards, to the secretary of the Institute 60 days prior to the opening of the annual convention at which they are to be presented. The secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual Proceedings, next issued as Tentative Standards. At a subsequent annual convention, they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Insti-

tute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of those voting shall vote in the negative.

ARTICLE VI

Amendments

Sec. 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

A. C. I. STANDARDS

Titles of current Standards, and of Tentative Specifications and Proposed Specifications and Recommended Practice of the Institute and their serial designation are here listed with reference to their publication in the Proceedings of the American Concrete Institute.

Curb and Gutter

Tentative Specifications for Concrete Curb and Concrete Curb and Gutter, S 6-E-27T. Vol. 23, p. 684.

Masonry Units

Standard Specifications for Concrete Building Block and Concrete Building Tile, P 1-A-29. Vol. 23, p. 696—revised—see tentative amendment, Vol. 24, p. 834. Adopted as standard by letter ballot. Note: Vol. 25, p. 605.

Standard Specifications for Concrete Sewer Manhole and Catch Basin Block, P 1-C-29. Vol. 23, p. 694. Adopted as standard by letter ballot. Note: Vol. 25, p. 605.

Standard Specifications and Building Regulations for Concrete Staves, P 4-A-26. Vol. 22, p. 666.

Tentative Specifications for Concrete Brick, P 1-B-26T. Vol. 21, p. 604—revised—see tentative amendment, Vol. 22, p. 667—and Note: Vol. 23, p. 690.

Tentative Recommended Practice for the Manufacture of Concrete Building Block, Building Tile and Brick, P 6-A-25T. Vol. 21, p. 473.

Proposed Recommended Practice for the Manufacture of Concrete Building Block and Tile (progress report of Committee 708 presented for discussion with a view to superseding Tentative Recommended Practice for the Manufacture of Concrete Building Block, Concrete Building Tile and Brick, P 6-A-25T—see above). Vol. 27, p. 1001 (April 1931 JOURNAL).

Tentative Specifications for Cast Stone, P 3-A-29T. Vol. 25, 1929.

Proposed Recommended Practices in the Use of Cast Stone (Report of Committee 704, presented for discussion). Vol. 26, p. 760 (May 1930 JOURNAL).

Pipe and Drain Tile

Standard Specifications for Concrete Drain Tile, P 7-B-25. (As tentatively published Vol. 20, p. 678.)

Tentative Specifications for Reinforced Concrete Sewer Pipe, P 7-C-25T. Vol. 21, p. 584.

Tentative Specifications for Plain Concrete Sewer Pipe, P 7-A-24T. Vol. 20 p. 669.

Tentative Specifications for Reinforced Concrete Culvert Pipe. (Second report. Adopted by the Joint Concrete Culvert Pipe Committee representing American Concrete Institute—Com. J-2—American Association of State Highway Officials; American Concrete Pipe Association; American Railway Engineering Association; American Society for Testing Materials; American Society of Civil Engineers; U. S. Department of Agriculture.) Vol. 25, 1929.

Reinforced Concrete

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Tentative Construction Specification for Concrete work on Ordinary Buildings, 502-31T. Vol. 26, p. 1 (November 1929 JOURNAL); discussion Vol. 26, p. 580 (March 1930 JOURNAL); Vol. 27, p. 99 (September 1930 JOURNAL); amended and adopted as tentative specification Vol. 27, p. 1181 (May 1931 JOURNAL).

Tentative Construction Specification for Concrete Work on the Small Job, 506-31T. Vol. 27, p. 65 (September 1930 JOURNAL); discussion, Vol. 27, p. 525 (January 1931 JOURNAL); amended and adopted as tentative specification, Vol. 27, p. 1184 (May 1931 JOURNAL).

Tentative Specification for Supplying, Fabricating and Setting Reinforcing Steel on Ordinary Buildings, 503-31T, with an appendix: "A Steel Setters' Primer." Vol. 26, p. 444 (February 1930 JOURNAL); discussion Vol. 26, p. 910 (June 1930 JOURNAL); adopted as tentative specification Vol. 27, p. 1186 (May 1931 JOURNAL).

Roads

Standard Specifications for One-Course Concrete Pavement for Highways, S 6-A-28. Vol. 20, p. 695.

Standard Specifications for Two-Course Portland Cement Concrete Pavement for Highways, S 6-B-28. (As tentatively published Vol. 20, p. 710.)

Standard Specifications for One-Course Portland Cement Concrete Street Pavement, S 6-C-28. (As tentatively published Vol. 20, p. 716.)

Standard Specifications for Two-Course Portland Cement Concrete Street Pavement, S 6-D-28. (As tentatively published Vol. 20, p. 724.) (Tentative revisions adopted 1928. Vol. 24, p. 852.)

Sidewalks and Floors

Standard Specifications for Portland Cement Concrete Sidewalks, C 2-B-25. Vol. 21, p. 591.

Standard Specifications for Concrete Floors, C 2-A-24. Vol. 20, p. 739.

Proposed Recommended Practice for Heavy Duty (concrete) Floor Finish with Notes on Light Duty Floor Finish
and

Proposed Recommended Practice for Dusted-on (concrete) Floor Finish (embodied in report of Committee 802 presented for discussion) Vol. 26, p. 520 (March 1930 JOURNAL) and discussion Vol. 27, p. 115 (September 1930 JOURNAL).

Surfaces

Standard Recommended Practice for Portland Cement Stucco. C 3-A-23. Vol. 19, p. 471.

Tentative Recommended Practice for Treatment of Exterior Surfaces of Industrial Reinforced Concrete Buildings, C 3-B-25T. Vol. 21, p. 564.

Tentative Specifications, Finish Coat, Portland Cement Stucco, C 3-C-29T. Vol. 25, 1929.

Proposed Recommended Practice for the Use of Pigment Admixtures in Troweled Concrete Surfaces—progress report of Committee 408, presented for discussion, Vol. 27, p. 975 (April 1931 JOURNAL).

Miscellaneous

Standard Methods for the Measurement of Concrete Work, C 5-A-26. Vol. 22, p. 655.

Tentative Purchase Specifications for Concrete Aggregates, E 5-A-29T. Vol. 25, 1929.

Standard Specifications for Monolithic Concrete Sewers and Recommended Rules for Sewer Design, S 3-A-24. Vol. 20, p. 757.
Standard Definitions, G 4-A-23. Vol. 19, p. 319.

Tentative Specification for Ready-Mixed Concrete, 504-31T. Vol. 27, p. 1177 (May 1931 JOURNAL).

Proposed Specification for Concrete Burial Vaults (progress report of Committee 709, presented for discussion) Vol. 27, p. 1251 (June 1931 JOURNAL).

The Institute has done other work in standardization involving specifications and recommended practice now regarded as obsolete or not brought to date, or progress reports which are incomplete and for these reasons not here listed.

MEDALS AND AWARDS

The Wason Medals

Founded by Leonard C. Wason, Boston, past-president, American Concrete Institute.

AWARDS, 1916-1929

For the most meritorious paper presented at each annual convention.

1916 Paper—A. B. MCDANIEL, "Influence of Temperature on the Strength of Concrete."

1917 Paper—CHARLES R. GOW, "History and Present Status of the Concrete Pile Industry."

1918 Paper—DUFF A. ABRAMS, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Paper—W. A. SLATER, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Paper—W. A. HULL, "Fire Tests of Concrete Columns."

1921 Paper—H. M. WESTERGAARD, "Moments and Stresses in Slabs."

1922 Paper—GEORGE E. BEGGS, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gauges."

1923 Paper—JOHN J. EARLEY, "Building the Fountain of Time."

1924 Two Papers—RICHARD L. HUMPHREY, "Twenty Years of Concrete" and "The Promise of Future Development."

1925 Paper—E. A. DOCKSTADER, "Report of Tests Made to Determine Temperatures in Reinforced-Concrete Chimney Shells."

1926 Paper—A. BURTON COHEN, "Correlated Considerations in the Design and Construction of Concrete Bridges."

1927 Paper—ARTHUR R. LORD, "Notes on Concrete—Wacker Drive, Chicago."

1928 Paper—FRANKLIN R. MCMILLAN, "Concrete Primer."

1929 Paper—L. G. LENHARDT, "The Concrete Lining of Detroit Water Tunnels."

1930 Paper—I. E. BURKS, "Concreting Methods at the Chute a Caron Dam."

AWARDS, FOR RESEARCH, 1929, 1930

S. C. HOLLISTER, for advancing the art of bridge construction in the design, construction and test of a concrete skew arch as reported in his 1928 paper: "The Design and Construction of a Skew Arch."

H. F. GONNERMAN and P. M. WOODWORTH for the work reported in their 1929 paper: "Tests of Retempered Concrete."

The Turner Medal

Founded 1927 by Henry C. Turner, New York, past-president, American Concrete Institute, a gold medal will be awarded not oftener than once a year "for notable achievement in or service to the concrete industry."

AWARDS, 1928, '29 AND '30

Awarded, 1928, to ARTHUR N. TALBOT, for "outstanding contributions to the knowledge of reinforced concrete design and construction."

Awarded, 1929, to WILLIAM K. HATT, for "pioneer work in reinforced concrete research; for a quarter century of devoted, outstanding and continuous service in developing the knowledge of concrete."

Awarded, 1930, to F. E. TURNEAURE, for "distinguished service in formulating sound principles of reinforced concrete design."

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An asterisk (*) designates a Supporting Member

Diligent effort is made to keep the official membership card list corrected to date in every detail. This printed list is compiled from that with corrections to July 1, 1931. This register is cross-indexed to include names of individuals designated as representatives of corporation memberships. If errors have crept in please notify us promptly. Names and addresses of new members will be published in the JOURNAL.

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Bird, Harold C. (Dee Duke University Library)
*Bird, Paul P. (See Boston Sand & Gravel Co.)
Birkland, George K., 6054 Pershing Ave., St. Louis, Mo.
Birmingham Public Library, Birmingham, Ala. (Lila May Chapman, Director)
Birmingham Slag Co., Age Herald Bldg., Birmingham, Ala.
Birnie Sand & Gravel Co., No. Wilbraham, Mass. (J. E. Bowker)
Bishop, Harry, c/o P. W. D., Sandakan (via Singapore), British N. Borneo
Bishop Estate, B. P., P. O. Box 3466, 848-850 Kaahumanu St., Honolulu, T. H. (Theo. B. Bush)
Bishop, Howard, Sharon Steel Hoop Co., Sharon, Pa.
Bissell, Clinton T., 85 John St., New York, N. Y.
Bjuke, Captain N. Rob, c/o Kwangtung River Conservancy Commission, Canton, South China
Black Co., John H., 505 Delaware Ave., Buffalo, N. Y.
Blakeslee & Sons, Inc., C. W., 58 Waverly St., New Haven, Conn. (W. T. Gilbert)
Blackmore, G. C. (See Northwestern States Portland Cement Co.)
Black & Veatch, Mutual Bldg., Kansas City, Mo. (N. T. Veatch, Jr.)
Blackwood, W. H. (See Hud-Cin Building Products, Inc.)
Blair, Wm. R. (See Southwestern P. C. Co.)
Blake, F., "Winsley House" 1, Cotele Nillas, Stoke, Plymouth, Devon, England
Blake, H. C., 1750 Strathcona Drive, Detroit, Mich.
Blancato, Virgilis, N. Y. Imitation Lime & Granite Stone Co., 1218 Oak Point Ave., Bronx, N. Y.
Blanchard, Arthur H., 12 Dennison Building, 515½ Jefferson Ave., Toledo, Ohio
Blanchard & Crandell, 38 5th Ave., Saratoga Springs, N. Y. (A. E. Blanchard)
Blanchard, A. E. (See Blanchard & Crandall)
Blanchard, H. A. (See Buffalo Wash Tray Works)
Blanchard, F. E., 2706 Terrace Ave., Columbus, O.
Blanchard, J. E., Director of Public Works, City of Montreal, Que., Canada
Blank, Alton J., Apartado Num 232, Puebla, Estado de Puebla, Mexico.
***Blaw-Knox Co.,** Box 915, Pittsburgh, Pa.
*Block, H. C. (See Missouri Portland Cement Co.)
Blue Jay Concrete Products Co., 2010 Pennsylvania Ave., Madison, Wisc. (Charles Clare)
Blue Ridge Talc. Co., Inc., Henry, Va. (C. O. Kitson, Secy.-Treas.)
Blyth, Charles E., c/o Chas. Nelson & Co., Stockton Works, Rugby, England
Boettcher, Henry (See Lancaster Concrete Tile Co.)
Bogk, Frederick C. (See Ricketson Mineral Color Works)
Bogue, Robert H., Bureau of Standards, Washington, D. C.
Bolton Pratt Construction Co., The, 426 Marion Bldg., Cleveland, Ohio (K. H. Pratt)
Bond, Guillermo, 11 de Setiembre 1409, Buenos Aires, Argentina
Boomer, Roy E., 1541 W. Boston Blvd., Detroit, Mich.
Borchard, E. K. (See Universal Atlas Cement Co.)
Borut, J., C. E., c/o Public Works Dept., Jerusalem, Palestine
Bose, N. N., District Engr., The Concrete Association of India, Bank Buildings, Chandni Chowk, Delhi, India
Boston Building Department, 901 City Hall Annex, Boston, Mass. (Edw. W. Roemer, Comm.)
***Boston Sand & Gravel Co.,** 120 First St., Cambridge, Mass. (Paul P. Bird)
Boston Transit Mixers, Inc., 104 Forsythe St., Boston, Mass. (John T. McMorrow)

Boswell, H. C. (See L. C. Everist, Inc.)
Boulanger, J. L. (See Quebec Dept. of Roads)
Bound Brook Crushed Stone Co., 9 Clinton St., Newark, N. J. (L. Upton)
Bourne, C. L., c/o Geo. Rackle & Sons, Station D, Cleveland, Ohio
Bowker, J. E. (See Birnie Sand & Gravel Co.)
Bowling, R. M. (See The Bettendorf Co.)
Bowman, H. L., Prof. of Civil Engineering, Drexel Institute, Philadelphia, Pa.
Boyer, E. D., 55 E. 10th St., New York, N. Y.
Boyle, Ernest E., 901 24th Ave., North, Seattle, Wash.
Brabant, Harry M., 4 Southampton Row, London, W. C. 1., England
Bradbury, R. D., 1049 National Press Bldg., Washington, D. C.
Braden, A. B. (See Columbus Testing Laboratories, Inc.)
Bradford, L. (See Broken Hill Proprietary Co., Ltd.)
Bradley, C. M. (See Youngstown Sheet Tube Co.)
Bragg, J. G., 15 S. Third St., Easton, Pa.
Bragger, E. Y., Shawomet, R. I.
Branda, Christopher, 97 Hazelwood Ave., Bound Brook, N. J.
Brandes, Raymond L., 426 Beech St., Arlington, N. J.
Brandtzaag, Anton, A/S Betongbygg, Trondhjem, Norway
Brassert, Walter, 2336 Edgewood Pl., Kalamazoo, Mich.
Bray, Thorpe (See Concrete Constructions, Ltd.)
Brazer, George H., 79 Milk St., Boston, Mass.
Breitengross, R. A., Box No. 5, Buffalo, Ia.
Brekke, G. N., 1111 S. West Ave., Johnson City, Tenn.
Brennan, Frank X., 2507 Tasher St. Philadelphia, Pa.
Brett, John F., 662 Melrose Ave. Montreal, Que., Canada.
Brickett, Edw. M., 819 Linden St., Allentown, Pa.
Bridgeport Stone Co., The, 1106 North Ave., Bridgeport, Conn. (Frank E. Clark, Treas.)
Bridges, G. P., c/o Simon Carves Ltd., 20 Mount St., Manchester, England.
***British Columbia Cement Co., Ltd.**, Victoria, B. C., Canada (Edwin Tomlin, Director, Treas.)
British Columbia Elec. Ry. Co., Ltd., Vancouver, B. C., Canada (E. E. Carpenter)
Britt, Harris (See Sinclair & Grigg)
Brock, Arthur S., 211 Seventh St., Hinsdale, Ill.
Brockway, R. R., 616 6th St. S. E., Minneapolis, Minn.
Broe, Harald, c/o Christiani & Nielsen, Post-box 120, Rio de Janeiro, Brazil
Broga, George F., Onondaga Litholite Co., 102 N. Beech St., Syracuse, N. Y.
Broken Hill Proprietary Co., Ltd., Box 196, Newcastle, N. S. W., Australia (L. Bradford, Mgr.)
Brooke, Howard (See Bergen Building Block Co.)
Brooks, Ernest W., The Rugby Portland Cement Co., Bilton Works, Rugby, England
Brooks, Robert M., 178 Washington Ave., Bellville, N. J.
Brooks, W. E., Municipal Bldg., Morgantown, W. Va.
Brown, Burtis, 65 Franklin St., Boston, Mass.
Browne, E. F., 1025 Santa Fe Bldg., Dallas, Texas
Brown, F. M. (See American Sewage Disposal Co., Inc.)
Brown, John A. W., 17 Main Street East, Hamilton, Ont.
Brown, J. F., 3426 E. 89th St. Chicago, Ill.
Browne, Nolan (See Penniman Concrete & Material Co.)
Brown, Paul G., Philadelphia Bank Bldg., Philadelphia, Pa.
Brown, Philip P., 1101 Bekins Bldg., Vancouver, B. C., Canada
Brown, Robert (See St. Joseph Cinder Block Co.)
Brown, Rex L., 300 Laboratory of Applied Mechanics, University of Ill., Urbana, Ill.
Brown, R. L., 85 East Gay St., Columbus, Ohio
Brown, S. N. (See Consumers Supply Co.)
Brown & Myers, 1721 Petroleum Bldg., Oklahoma City, Okla.
Browne, Walter E., 1028 1st Wis. Nat'l Bank Bldg., Milwaukee, Wis.
Brownell, Parker J., 618 Fulton St., Carthage, N. Y.
Brown, H. Whitemore, Concord, Mass.
Bruce, W. C. (See Barnsdall Tripoli Co.)
Brumbaugh, W. V. (See National Lime Association)
Brundage, Avery, 110 S. Dearborn St., Chicago, Ill.
Bruner, Louis S., 20 So. 36th St., Philadelphia, Pa.
Brunnier, H. J., 612 Sharon Bldg., San Francisco, Cal.
Bryant, Charles B., c/o States Road Commission, Federal Reserve Bank Bldg., Baltimore, Md.
Bryant, Henry F., 334 Washington St., Brookline, Mass.
Buck, R. C., U. S. National Bank Bldg., Superior, Wis.
*Buente, C. F. (See Concrete Products Co. of America)
Buerkin, Julius A. (See Buerkin & Buerkin)
Buerkin & Buerkin, 201 Broadway, Quincy, Ill. (Julius A. Buerkin)
Buffalo Art Stone Corp., 449 Center St., Lackawanna, N. Y. (E. H. DeVoe)
Buffalo Litholite Company, Box 12, Lackawanna, N. Y. (George E. Hipp)
Buffalo Slag Company, 1070 Ellicott Square, Buffalo, N. Y. (Carlton S. Wicker, Mgr.)
Buffalo Wash Tray Works, Inc., 601 Tonawanda St., Buffalo, N. Y. (H. A. Blanchard)
Buhler, Adolf, Chief Engineer of Bridges, National Swiss Railways, Berne, Switzerland
Builders Concrete Stone Co., State Park Ave., Pawtucket, R. I. (Fred D. Fuller)
Bulger, John, 368 Sherman St., Akron, Ohio
Bullen, Carroll A., 111 W. Washington St., Chicago, Ill.
Bundi Portland Cement, Ltd., Lakheri R. S. (B. B. & C. I. Rly.), Rajputana, India (E. Christensen, Mgr.)
Bunker, Geo. C., P. O. Box 5035, Ancon, Canal Zone
Burchartz, Prof. H., Staatliches Materialprufungsamt, Unter den Eichen 87, Berlin, Dahlem, Germany
Burd, Edw. M. (See Stevens & Wood)
Burford, John Nicholas, 480 Vermont St., Buffalo, N. Y.
Burgar, F. A., Box 200, Campbellford, Ont., Canada
Burge, A. W., 4059 Beaconsfield Ave., Montreal, P. Q., Canada
Burger, Alfred A., 1149 Leader Bldg., Cleveland, Ohio
Burgess, C. C., 6842 Thomas Blvd., Pittsburgh, Pa.
Burgess, L. T., "Marlowe", Higher Road, Halewood, nr. Liverpool, England
Burgess, Philip, Burnip Construction Co., Inc., 568 E. Broad St., Columbus, Ohio
Burggraf, Fred, Highway Research Board, 21st & B Streets, Washington, D. C.
Burke, Charles H., 342 Madison Ave., New York, N. Y.

Burke, John A., New England Concrete Pipe Corp., 79 Milk Street, Boston, Mass.
Burke, J. E., The Ready Mixed Concrete Co., 27 Barbeau St., Pittsburgh, Pa.
Burke, J. M., 406 1st Nat'l Bank Bldg., Stockton, Calif.
Burks, I. E., 2400 Oliver Bldg., Pittsburgh, Pa.
Burmeister, R. A., 1570 So. 15 Place, Milwaukee, Wis.
Burns, Edward (See Guarantee Construction Co.)
Burns, Homer S., Freeport, Texas
Burns, Inc., D. F. & W. G., 9 Park St., Boston, Mass. (Walter G. Burns)
Burns & McDonnell Engineering Co., 406 Interstate Bldg., Kansas City, Mo. (C. S. Timanus)
Burns, Walter G. (See D. F. & W. G. Burns, Inc.)
Burr, Henry A., 3515 Richland Ave., Nashville, Tenn.
Burrell Engineering & Constr. Co., 1 North Canal St., Chicago, Ill. (Joseph Wilson, Secy.)
Burrell, H. P., MacArthur Concrete Pile Corp., 19 West 44th Street, New York, N. Y.
Burrington, Jr., M. J., Main St., Bennington, Vt.
Burton, Frank, 1736 Dime Bank Bldg., Detroit, Mich.
Bush, Theo. B. (See B. P. Bishop Estate)
Bushnell, E. B. (See Masonite Corp.)
Bushnell, H. L., 1501 Dexter Horton Bldg., Seattle, Wash.
Busse, F. A., 871 Shrine Bldg., Memphis, Tenn.
Buswell, J. M. (See San Joaquin Light & Power Corp.)
Butler Bin Co., 154 Wisconsin Ave., Waukesha, Wis. (Morgan R. Butler, Pres.)
Butler, C. M. (See Marquette Cement Mfg. Co.)
Butler, D. B., 78 Marsham St., Westminster, S. W. 1, London, England
Butler, Gordon H. (See Polaris Concrete Products Co.)
Butler, Joe B., Assoc. Prof. of Civil Engineering, Mo. School of Mines, Box 547, Rolla, Mo.
Butler, Morgan R. (See Butler Bin Co.)
Butler, R. B., Box 1023, Bryan, Texas
Button, Harold, B. Sc., 17 Arncliffe Road, Harrogate, England
Byrne, Thos. S., 1005 Fort Worth National Bank Bldg., Fort Worth, Tex.
Cohen, A. B., 1 Madison Ave., New York, N. Y.
Caffiaux & Villeneuve, Chicoutimi, P. Q., Canada (P. A. Villeneuve)
Caiola, Fred, 2840 Wellman Ave., Bronx, N. Y.
Calhoun, Jr., John C., Texas Concrete Pipe Co., 1026 Temple, Houston, Texas
California Institute of Technology, Dept. of Civil Engineering, Pasadena, Calif. (R. R. Martel, Assoc. Prof.)
*__California Portland Cement Co.,__ Colton, Calif. (W. C. Hanna, Chief Chemist)
California, State of, Division of Highways, 3435 Serra Way, Sacramento, Calif. (T. E. Stanton)
California Stucco Products of N. E., 411 Walden St., Cambridge, Mass. (Napoleon M. Bernier)
California Stucco Products Co., 1344 University Ave., Rochester, N. Y. (H. D. Kneeland)
California Stucco Products Co. of N. C., 340 Dore St., San Francisco, Calif. (Percy R. Stuart)
Callier, B. C. (See Cement Gun Co., Inc.)
*__Calumet Steel Co.,__ 33 N. La Salle St., Chicago, Ill. (F. G. Carrel)
Cameron, E. H. (See Jackson & Moreland)
*Cameron, W. P. (See Olympic Portland Cement Co.)
Campbell, J. Bow, c/o Reinforcing Steel Co, P. O. Box 117, Salisbury, Rhodesia, S. Africa.
Campbell & Sons, Ltd., James, Creek St., Brisbane, Australia (J. D. Bell)
Campbell, John T., 813 Clark Bldg., Pittsburgh, Pa.
Campbell, R. D. (See The General Cement Products Co.)
Campbell, S. A., 251 Elmdorf Ave., Rochester, N. Y.
Campion, H. T., 112 So. 16th St., Philadelphia, Pa.
Campion, William L., 76 Moraine St., Jamaica Plain, Mass.
*__Canada Cement Co., Ltd.,__ Montreal, Que. (H. S. Van Scoyoc)
Canada Crushed Stone Corp., Sun Life Bldg., Hamilton, Ont., Canada (C. M. Doolittle)
Canadian Engineering Standards Assn., 178 Queen St., Ottawa, Canada (B. Stuart McKenzie, Secy.)
Canadian National Parks Branch, Dept. of the Interior, Ottawa, Canada. (J. B. Harkin, Commissioner)
Canadian National Railways, Montreal, Que., Canada (W. A. Duff)
Canton Block & Tile Co., Canton, S. Dak. (E. D. Rowe)
Capital Avenue Cement Works, Box 5004, St. Paul, Minn. (A. O. Hovde)
Capouch, M. E., 208 S. La Salle St., Chicago, Ill.
*__Carey Co., Philip,__ Lockland, Cincinnati, O. (C. V. R. Fullenwider, Mgr. Elastite Dept.)
Carlock, J. B. (See Jones & Laughlin Steel Corp.)
Carlon Construction Co., Oskaloosa, Ia. (Harry F. Carlon)
Carlon, Harry F. (See Carlon Construction Co.)
Carlson, Roy W., P. O. Box No. 33, Azusa, Calif.
Carmel, Jos. Ed., Supt. of Bldgs., City Hall, Montreal, Canada
Carnwath, James (See Independent Concrete Pipe Co., Ltd.)
Carpenter, C. A. (See Colorado State Agricultural College)
Carpenter, E. E. (See British Columbia Elec. Ry. Co., Ltd.)
*Carrel, F. G. (See Calumet Steel Co.)
Carrick, T. Bright, Consulting Engineer, 806 North Main St., High Point, N. C.
Carter, Wilfred E., 186 Pond St., Natick, Mass.
Carty, Bruce F. (See The Maul Co.)
Carvalho, Manfredo de A., Rua Santa Alexandrina 146, Rio de Janeiro, Brazil
Case School of Applied Science, Cleveland, O. (F. H. Neff)
Casey Co., John F., P. O. Box 1753, Pittsburgh, Pa. (Samuel Leslie Fuller)
Cassell, J. D., 2008 Walnut St., Philadelphia, Pa.
Cast Concrete Products Assn., Ltd., The, 20 Dartmouth St., S. W. 1., London, England (A. S. Windsor)
Catts, E. C. (See J. B. McCrary Engineering Corp.)
Cavanagh, Jr., W. Curran, Box 1878, Port of Seattle, Seattle, Wash.
Caye Construction Co., Inc., 356 Fulton Street, Brooklyn, N. Y. (Webster J. Caye, Pres. and Treas.)
Caye, Webster J. (See Caye Construction Co., Inc.)
Ceglarek, Wallace, Clifford Hotel, Detroit, Mich.

Cement Gun Co., Inc., Allentown, Pa. (B. C. Callier)
Cement Gun Construction Co., 537 So. Dearborn St., Chicago, Ill. (John V. Schaefer, Pres.)
Cement Products Co., P. O. Box 935, Birmingham, Ala. (G. L. Cox, Pres.)
Cementproducenternas, 1 Finland Forening, Kalevagatan 20, Helsingfors, Finland (Urho Aberg)
Cement Stone & Supply Co., 1447 N. Meade, Wichita, Kansas (B. F. Krehbiel, Mgr.)
Central Concrete Mixing Corp., 527 Smith St., Brooklyn, N. Y. (A. Johnson)
Central Texas Iron Works, Drawer L 11, Waco, Texas (L. H. Wood)
Chace, Clyde B., P. O. Box 186, West Palm Beach, Fla.
***Chain Belt Co.,** 736 Park St., Milwaukee, Wis. (B. F. Devine)
Chamier, A. U., Division of Engineering, 8100 W. Warren Ave., Dearborn, Mich.
Champe, Geo., 1025 Nicholas Bldg., Toledo, Ohio
Champion, E. C., c/o Lone Star Cement Co., Nazareth, Pa.
Chandler, J. M. (See Oklahoma Glazed Cement Pipe Co.)
Chapman, Cloyd M., 105 W. 40th St., New York City, N. Y.
Chapman, John J., 702 Westfield, Elizabeth, N. J.
Chapman, Lila May (See Birmingham Public Library)
Chapman & Oxley, 372 Bay St., Toronto, Ont. (J. Morrow Oxley)
Charron, Louis, 3125 Harrison Ave., Detroit, Mich.
Cheney, J. Tyrell (See Kraus Process, Incorporated)
Chhapgar, F. K., 627 Parsi Colony, Dadar, Bombay, India
Chiao Tung University, Library, College of Civil Engineering, Tangshan, North China
Chicago Architectural Stone Co., 1301 First Ave., Maywood, Ill. (M. A. Williams, Vice-Pres.)
Chicago Art Marble Co., 2883-87 Hillock Ave., Chicago, Ill. (M. S. Marcus, Pres.)
Chittenden, Howard L., East Main St., Clinton, Conn.
Choudhury, A. C., 21/1A Shyamananda Road, P. O. Kalighat, Calcutta, India.
Christensen, Einar, Consulting Engr., 122 East 42nd St., New York, N. Y.
Christensen, E. (See Bundi Portland Cement, Ltd.)
Christensen, E., Mgr. Bundi Portland Cement, Ltd., Lakheri R. S. (B. B. & C. I. Rly.), Rajputana, India
Christiani & Nielsen, Raadhusplads 77 Copenhagen, Denmark
Christiania Portland Cementfabrik, Christiania, Oslo, Norway (Harald Jakhelln)
Christie, H. A., 31 Washington St., East Orange, N. J.
Christoffel Art Stone Co., Station F, No. Milwaukee, Wisc. (John Christoffel)
Christoffel, John (See Christoffel Art Stone Co.)
Christy, C. O. (See H. C. Nutting Co.)
Chubb, Joseph H., 521 5th Ave., New York, N. Y.
Churchill, H. D., Case School of Applied Science, Cleveland, Ohio
Chutter, H. W. (See Jourdan Concrete Pipe Co.)
Ciampa, Felix A., c/o E. E. Seelye, 101 Park Ave., New York, N. Y.
Cimentaries et Briquetaries Reunies C. B. R., 117 Avenue De France, Antwerp, Belgium (F. Van Ortroy, Secretary)
Cincinnati Builders Supply Co., The, 209 E. Sixth Street, Cincinnati, Ohio (Juluis J. Warner)
Cincrete Corporation, 591 Canal St., Milwaukee, Wisc. (L. E. Pitner)
Cinder Block & Material Co., P. O. Box 137, Cedar Rapids, Iowa (H. L. Spaight)
Cinder Block Corporation, Kate Ave. and W. M. R. R., Baltimore, Md. (G. W. Creighton, Gen. Mgr.)
Cinder Concrete Units Corp., 200 Devonshire St., Boston, Mass. (Stephen Kearney)
Cinder Products, Inc., Carthage, Cincinnati, Ohio (W. Keith Clark)
Cisneros, Raul, Avenida Colon 150, Cordoba Argentina
Cissel, James Harlan, 225 Engineering Bldg., University of Michigan, Ann Arbor, Mich.
Clair, Miles N. (See Thompson-Lichtner Co., Inc.)
Clapper, Lyle, 304 Construction Bldg., Dallas, Texas
Clarahan, Jr., Charles H., 649 Hillside Ave., Glen Ellyn, Ill.
Clare, Charles (See Blue Jay Concrete Products Co.)
Clark, B. E., 402 First Nat. Bank Bldg., Oklahoma City, Okla.
Clark, Charles A., Pres., Clark & Burrows, Inc., 3605 Gillon Ave., Dallas, Texas
Clarke, Frank E. (See The Bridgeport Stone Co.)
Clark, Kenneth M., 3645—16th Ave. So., Minneapolis, Minn.
Clark, W. B., Lehigh Portland Cement Co., Bankers Bond Bldg., Birmingham, Ala.
Clark, W. Keith (See Cinder Products Inc.)
Claussen, Carlos F., Casilla 185, Valparaiso, Chile
Claussen & Claussen, Inc., 1109 Buyers Bldg., Portland, Ore. (W. E. Claussen, Sec.-Treas.)
Claussen, W. E. (See Claussen & Claussen, Inc.)
Clay Products Institute of Calif., 569 Chamber of Commerce, Los Angeles, Calif. (Norman W. Kelch)
Clay, Wharten (See Associated Metal Lath Mfgrs.)
Clement, R. B., Engr., Refiners Oil Co., 315 Miami Sav. Bank Bldg., Dayton, Ohio
Clemmer, H. F., Engineer of Tests & Materials, Engineer Dept., Dist. of Columbia, Washington, D. C.
Clicquennoi, I. M., 204 Wisconsin Ave., Milwaukee, Wis.
Cline, L. A., 1013 North Sycamore St., Los Angeles, Calif.
Clinton Metallic Paint Co., Clinton, N. Y. (John Mott, Vice-Pres.)
Clos, Charles, 227 East 126th St., New York, N. Y.
Clough, G. C., c/o F. T. Ley Co., Inc., 1215 Main St., Springfield, Mass.
Clousing, Louis, 213 City Hall, Minneapolis, Minn.
Clutz, Frank H., 159 Broadway, Gettysburg, Pa.
Coale, George B. (See J. D. Loizeaux Lumber Co.)
Cobb, L. A. (See Billingham & Cobb)
Cobb, W. G., 347 Madison Ave., New York, N. Y.
Coddington, Samuel C. (See Coddington Engineering Corp.)
Coddington Engineering Corp., 290 Third St., Milwaukee, Wis. (Samuel C. Coddington)
Coffman, Herbert, Hercules Cement Corp., 1700 Walnut St., Philadelphia, Pa.
*Coghlan, R. R. (See Marquette Cement Manufacturing Co.)

Cognets & Co., Louis des, 413 E. Main St., Lexington, Ky. (E. R. Ackerman, Secy.-Treas.)
Colburn, D. S., c/o Marquette Cement Mfg. Co., 140 S. Dearborn St., Chicago, Ill.
Colby, Edwin W., 79 Milk St., Boston, Mass.
Collier, Ira L., 6019 34th Ave., N. E., Seattle, Wash.
Collins, H. G., 247 Kenworth Rd., Columbus, Ohio
Collings Co., W. A., 937 7th St., Santa Monica, Calif. (W. A. Collings, Mgr.)
Collings, W. A. (See W. A. Collings Co.)
Colloy Production Company, 4030 Chouteau Ave., St. Louis, Mo. (C. P. Derleth)
Colmar, Daniel, Ramloc Stone Co., Inc., 1055 Broadway, Albany, N. Y.
Colorado State Agricultural College, Road Materials Testing Laboratory, Colo. Experiment Station, Fort Collins, Colo. (C. A. Carpenter)
Colorway, Inc., 3800 S. Racine Ave., Chicago, Ill. (Marshall R. Rust)
Columbus Testing Laboratories Inc., 755 N. High St., Columbus, Ohio (A. B. Braden, Pres.)
Columbia University, Dept. of Civil Engineering, New York, N. Y. (A. H. Beyer, Director of Tests)
Colwell, Curtis C., Assistant County Engineer, Essex County, Hall of Records, Newark, N. J.
Comite Para Propagar el Uso del Cemento Portland, Gante No. 1, Mexico City, Mexico
Commonwealth Portland Cement Co., 4 O'Connell St., Sydney, Australia (Fred J. Beardmore, Secy.)
Conahey, George c/o Johns-Manville Corp., 292 Madison Ave., New York, N. Y.
Concrete Bldg. Units Co., 1129 Garfield St., Oak Park, Ill. (P. F. Huxhold, Secy.)
Concrete Association of India, P. O. Box No. 138, Bombay, India (Alan Moncrieff)
Concrete Block & Products Co., 5740 Martin Ave., Detroit, Mich. (Jos. A. Sattler)
Concrete Constructions Ltd., Broughton House, 47 King St., Sydney, Australia (Thorpe Bray)
Concrete Construction Pty., Ltd., Collins House, 360 Collins St., Melbourne, Australia (L. A. Deegan)
Concrete Engineering Service Co., 230 W. 9th Ave., Columbus, Ohio (H. C. McCall, Ch. Engr.)
Concrete Marble Co., 5300 McKissock Ave., St. Louis, Mo. (G. C. Turner)
Concrete Products Ltd., 5 Banner Block, Regina, Sask.
*__Concrete Products Co. of America,__ Diamond Bank Building, Pittsburgh, Pa. (C. F. Buente)
Concrete Protection & Color Co., 1312 N. Marston St., Philadelphia, Pa. (John McKay)
Concrete Publishing Co., 400 W. Madison Ave., Chicago, Ill. (Norman M. Stineman)
Concrete Reinforcing Steel Institute, 1207 Tribune Tower, Chicago, Ill. (R. W. Johnson)
Concrete Specialties Co., P. O. Box 330, Camden, N. J. (L. A. Goodwin, Secy.)
*__Concrete Steel Co.,__ 2 Park Ave., New York, N. Y. (J. F. Havemeyer)
*__Concrete Steel Co.,__ 2 Park Ave., New York, N. Y. (James F. Curley)
Concrete Steel Engineering Co., 1902 Park Row Bldg., New York, N. Y. (Wm. Mueser)
Concrete Steel Fireproofing Co., 1245 Michigan Theatre Bldg., Detroit, Mich. (S. V. Taylor, Pres.)
Condit, G. H., 501-2 Mutual Home Bldg., Dayton, Ohio
Condit, W. E. (See The Henry J. Spieker Co.)
Condron & Post, 53 Jackson Blvd., W., Chicago, Ill. (C. L. Post)
Cone, R. G., 17 High St., Moorestown, N. J.
Congleton Co., W. T., Box 327, Lexington, Ky. (W. T. Congleton)
Congleton, W. T. (See W. T. Congleton Co.)
Conklin, James Arthur, DeBaun Ave., Ramsey, N. J.
*Conn, Charles F. (See Giant Portland Cement Co.)
Connar, V. N., 516 Frick Bldg., Pittsburgh, Pa.
Connecticut State Highway Dept., Testing Laboratory, Portland, Conn. (F. G. Flood)
Connecticut Quarries Co., Inc., The, 152 Temple St., New Haven, Conn. (A. L. Worthen)
Connell, W. H. (See Pennsylvania State Highway Dept.)
Connors Steel Co., 1007 Empire Bldg., Birmingham, Ala. (Geo. W. Connors)
Connors, Geo. W. (See Connors Steel Co.)
Conrades, Otto S. (See St. Louis Material & Supply Co.)
Consolidated Cement Corp., 111 West Monroe St., Chicago, Ill. (A. W. King)
*__Consolidated Concrete Machinery Co.,__ Adrian, Mich. (Eugene F. Olsen, Vice-Pres.)
Consolidated Materials Corp., 605 Terminal Bldg., Rochester, N. Y. (Henry F. Marsh)
Consolidated Pipe Co. Ltd., Cor. Decarie Blvd. & C. P. R. Tracks, Cote St. Luc, Montreal, Que., Canada (E. A. Sherrard)
Consumers Supply, 42nd & State Sts., Milwaukee, Wis. (S. N. Brown)
Converse & Co., Inc., J. B., Mobile, Ala.
Conwell & Co., E. L., 2024 Arch St., Philadelphia, Pa. (E. L. Conwell)
Conwell, E. L. (See E. L. Conwell & Co.)
Cooke, C. E. (See Kalman Floor Co.)
Cook, F. W., 63 Eugene St., Buffalo, N. Y.
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Cooper, Hugh L. (See Hugh L. Cooper & Co., Inc.)
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Cooper, Nelson, 103 E. 125th St., New York, N. Y.
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Corridon, J. B., Clarendon Road, Penn Valley (Marbeth P. O.) Pennsylvania
Cory, Russell G. 50 Church St., New York City, N. Y.
Cothran, F. H., Beauharnois Construction Co., Beauharnois, P. Q., Canada
Cotter, Lt. Comdr. C. H., Navy Yard, Puget Sound, Wash.
Cotterell, Ernest L., Tarmac Limited, Ettingshall, Wolverhampton, England
Cousins, H. E. (See Lockwood Greene Engineers, Inc.)
Cote, A. U., 26 Queen St., E., Toronto, Ont., Canada
Covell Corporation, The, 1600 Walnut St., Philadelphia, Pa.

Cowell Lime & Cement Co., No. 2 Market St., San Francisco, Calif.
Cowin & Co., Ltd., Pacific & Yeoman Sts., Winnipeg, Manitoba (H. B. Henderson)
Cowper Co., The John W., Fidelity Bldg., Buffalo, N. Y. (John W. Cowper)
Cowper, John W. (See John W. Cowper Co.)
Cox, G. L. (See Cement Products Co.)
Cox, H. R. (See Assoc. of Portland Cement Mfgrs. Ltd.)
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Cranford, Inc., Frederick L., 149 Remsen St., Brooklyn, N. Y. (F. L. Cranford)
Crangle, W. R. (See Metropolitan Concrete Co.)
Creamer, Randall (See Frederick Snare Corp.)
Creighton, G. W. (See Cinder Block Corp.)
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Crume, W. H. (See Crume Brick Co.)
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Cummins, Charles A., 20 E. Franklin St., Baltimore, Md.
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Dahlquist, H. A. (See Perfected Burial Vault Cement Products Co.)
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Davis, J. F. (See Robert W. Hunt & Co.)
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Deroover, G. (See Groupement Prof. des Fabricante)
Deslauriers Column Mould Co., Inc., 101 Tremont St., Boston, Mass. (Henry A. Dahlen)

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Detroit Edison Company, 2000 Second Ave., Detroit, Michigan (A. S. Douglass, Construction Engineer)
Detroit Edison Co. Library, 2000 Second Ave., Detroit, Mich.
Detroit Public Library, Woodward & Kirby Aves., Detroit, Mich. (Adam Strohm, Librarian)
Detroit Testing Laboratory, The, 554 Bagley Ave., Detroit, Mich. (W. R. DeGowin)
Detroit, Library, University of, 6 Mile Road & Livernois, Detroit, Mich. (David P. Gilmore)
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Devine, P. S., Pharr, Texas
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Dewey Portland Cement Co., 315 Union Bank Bldg., Davenport, Ia. (K. P. Ferrell)
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Diehl, Inc., George C., 577 Ellicott Sq., Buffalo, N. Y. (Ray P. Diehl)
Diehl, Ray P. (See George C. Diehl, Inc.)
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Douglass, A. S. (See Detroit Edison Co.)
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Douthett, C. L. (See Waterloo Concrete Corp.)
Dow Chemical Co., Midland, Mich. (L. C. Stewart)
Dowd, Daniel M., 219 Walnut St., Springfield, Mass.
Downs, Allan B., 61 Elm St., Lebanon, N. H.
Doyle, Frank P., 309 W. 93rd St., New York, N. Y.
Drehmann Paving & Constr. Co., 521-523 Glenwood Avenue, Philadelphia, Pa. (C. E. Drehmann)
Drehmann, C. E. (See Drehmann Paving & Construction Co.)
Dresser, Jr., Theo. P. (See Abbott A. Hanks, Inc.)
Drew Co., Inc., Fred, 2525 Pennsylvania Ave., N. W., Washington, D. C. (Fred Drew, Pres. & Gen. Mgr.)
Drew, Fred (See Fred Drew Co., Inc.)
Dreyer, Walter, 245 Market St., San Francisco, Calif.
Driggers, Clyde, University Station, Gainesville, Fla.
Drill, Max, 889 Broad St., Newark, N. J.
Drysdale, W. F. (See Aerocrete Corp. of Canada, Ltd.)
Dube, Charles, 32 Cherrelyn St., Springfield, Mass.
Duff, C. M., University of Nebraska, Lincoln, Nebr.
Duff, W. A. (See Canadian National Railways)
Dufour, F. O. (See United Engineering & Const. Inc.)
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Duke University Library, Durham, N. C. (Harold C. Bird)
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Duntile Co. Ltd., 734 Canada Cement Bldg., Montreal, Que., Canada (R. McGillivray)
Duquesne Slag Products Co., Diamond Bank Bldg., Pittsburgh, Pa. (C. L. McKenzie)
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Dyer, Kenneth W. (See The Dyer Construction Co.)
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Eagle Chemical Co., 414 So. 3rd St., Milwaukee, Wis. (Charles Schrank, Secy.)
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Earle, Ltd., G. & T., Wilmington, Hull, England (G. F. Earle, Managing Director)
Earle, G. F. (See G. & T. Earle, Ltd.)
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Economy Concrete Company of Va., Inc., Box 1223, Richmond, Va. (Ernest Wiedemann)
Eddy, W. Paul, Onodaga Litholite Co., 102 N. Beech St., Syracuse, N. Y.
Eddystone Cement Co., 50 Howard St., New York, N. Y. (E. R. Wilner)
Edison Cement Co., New Village, N. J. (George G. Wilsnack)
Edwards, C. E. (See Lamar Pipe & Tile Co.)
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Eighmie, Borden C., R. F. D. No. 4, Schenectady, N. Y.
Eikel, E. (See Dittlinger Lime Co.)
Eisenmenger, P. E. (See Denton & Co.)
Eitzen, Henry R., North Glenside, Pa.
Eklund, H., Mrg., Hume Pipe Co. (Australia) Ltd., Brisbane, Queensland, Australia
Ekwall, A. (See Royal Swedish Board of Waterfalls)
Eldridge, H. W., New City, Rockland County, New York.
Eldridge, R. V. F., c/o Engineer's Office, 422 Collins St., Melbourne, Victoria, Australia
Electric Tamper & Equipment Co., 433 Railway Exchange Bldg., Chicago, Ill. (M. I. McCarty)
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Ellison and Russell, 821 Market St., San Francisco, Calif. (W. H. Ellison)
Ellison, W. H. (See Ellison & Russell)
Elmhurst-Chicago Stone Co., 400 W. 1st St., Elmhurst, Ill. (Martin Hammerschmidt)
Elms, Stephen F., 224 Cortland St., Apt. 22, Detroit, Mich.
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Emerson Co., The Sam W., 1836 Euclid Ave., Cleveland, Ohio (R. E. Laubscher)
Emerson-Norris Co., 163 North Beacon St., Brighton, Boston, Mass. (Frederick A. Norris)
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Federal Cement Tile Co., Hammond, Ind. (J. George Geyer, Gen. Supt.)
Federal Concrete Co., 667 Wyoming Ave., Buffalo, N. Y. (Walter E. Jones, Secy.)
Federici, Albert, Paterson, N. J.
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Fegles, D. B. (See Fegles Construction Co. Ltd.)
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***Ferro-Concrete Construction Co.,** 3rd and Elm Sts., Cincinnati, O. (W. P. Anderson)
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Fisher, Frank F., (See Fisher-DeVore Const. Co.)
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Fisher-DeVore Const. Co., The, Box 36, Evanston Sta., Evanston, Ohio (Frank F. Fisher)
*Fiske, Burton C. (See Fiske-Carter Construction Co.)
Fitch, T. A. (See Los Angeles City Engineering Dept.)
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Fleming, Paul K. (See American Concrete Marbelite Company,)
Flesheim, S. W. (See The Master Builders Co.)
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Fletcher-Thompson, Inc., P. O. Box 85, Bridgeport, Conn. (Edward A. Lambert)
Flight, Oscar, Carpenter St., Bendigo, Victoria, Australia
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Florida Portland Cement Co., P. O. Box 1075, Tampa, Fla. (Cyrus N. Wells)
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Fonseca, Humberto da, Escriptorio Technico da 52, Divisao, Sorocabana Ry., Sao Paulo, Brazil, S. A.
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Formigli, Paul (See Formigli Architectural Stone Co.)
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Foster, Harry L. (See The Foster Construction Co.)
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Foster, Winslow H., 9238 Pleasant Ave., Chicago, Ill.
Fougner, Hermann, 103 Park Ave., New York City
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Foundation Co. of Canada Ltd., The, 746 Sherbrooke St. West, Montreal, Que., Canada (V. G. Younghusband)
Fowden, Wm (See South Dakota Cement Plant)
Fowle, R. E. (See Granite Rock Co.)
Franc, Joseph, Bratislava, Ruzova 12, Czechoslovakia
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Franki, Pieux, 196 Due Gretry, Liege, Belgium
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Franklin, Jack, 7071 Plankinton Bldg., Milwaukee, Wisc.
Franklin Limestone Co., Nashville, Tenn. (A. B. Rodes)
*__Franklin Steel Works,__ Franklin, Pa. (E. E. Hughes, Vice-Pres.)
Franz, Herbert J., 30 Homes Ave., Dorchester, Mass.
Fraser, Andrew, 622-24 American National Ins. Bldg., Galveston, Texas
Fraser, Alexander, Department of Roads, Parliament Bldg., Quebec, Canada
Frauenfelder, Herman, 4600 Chippewa St., St. Louis, Mo.
Frazier, W. H. (See Pyramid Company)
Frech, H. E., Dist Engr., 1313 Syndicate Trust Bldg., St. Louis, Mo.
Freedman, J., c/o Mass. Cement Block Co., 909 Fellsway, Medford, Mass.
Freel, W. I., Testing Materials Lab., Purdue University, Lafayette, Ind.
Freeman, E. M., 414 Levy Bldg., Shreveport, La.
Freeman, G. L. (See Moran & Proctor)
Freeman, P. J., 519 Smithfield St., Pittsburgh, Pa.
Freibott, George B., Box 372, Port Monmouth, N. J.
French, R. M., Blue Ash, Ohio
Friberg, Bengt., c/o Laclede Steel Co., St. Louis, Mo.
Frick, Walter H., 824 Rebecca Ave., Wilkinsburg, Pa.
Fridstein, Meyer, 1601 Conway Bldg., Chicago, Ill.
Friebele, J. F., Broad St., Book Bldg., Trenton, N. J.
Friel, Francis S., c/o Albright & Mebus, 112 Easton Rd. S., Glenside, Pa.
Friel, G. M. (See Anchor Concrete Machinery Co.)
Friis, Kristen, Lokkevei 9, Oslo, Norway
Friis, Nelson A. (See Architectural Stone Co.)
Fritz Co., C. B., 138 N. Francis St., Madison, Wis. (N. J. Schmitz)
Froehling & Robertson, Richmond, Va. (T. B. Hartless)
Frolich, A. K., Louisville, Nebr.
Froemming Brothers Inc., 4384 N. Green Bay Ave., Milwaukee, Wis. (B. A. Froemming)
Froemming, B. A. (See Froemming Brothers Inc.)
Frosch, A. E., Eastern Ohio Sand & Supply Co., East Liverpool, Ohio
Frost, Chamberlain & Edwards, Slater Bldg., Woscester, Mass.
Frost, Hermann, A/S Jernbeton Trondhjem, Trondhjem, Norway.
Frost, Rikard V. (See Svenska Betongforeningen)
Fruchtbaum, J., c/o Reecon Inc., 37 Crestwood Ave., Buffalo, N. Y.
Fugate, G. L., Asssitant City Engineer, Dept. of Public Works, Houston, Tex.
Fulcher, Edmund, 404 22nd St., Brandon, Manitoba, Can.
Fullenwider, C. V. R. See The Philip Carey Company)
Fuller Co., Fuller Bldg., Catasauqua, Pa. (P. F. Stauffer, Sales Mgr.)
Fuller, Fred D. (See Builders Concrete Stone Co.)
Fuller, Geo. W. (See Fuller & McClintock)
Fuller, Samuel Leslie (See John F. Casey Co.)
Fullerton, Robert (See Made-Rite Products Co.)
Fuller & McClintock, 170 Broadway, New York City, N. Y. (Geo. W. Fuller)
Furlong, Irving, Bureau of Standards, 81 Appraisers Bldg., San Francisco, Calif.
Furphy, LeRoy F., 89-18 247th St., Bellerose Manor, Queens, L. I., N. Y.
Gabriel Steel Co., 13700 Sherwood Ave., Detroit, Mich. (W. F. Zabriskie)
Gage, Robert B., N. J. State Highway Dept., Box 106, Trenton, N. J.
Gain, Elmer W., 820 Baugh Ave., East St. Louis, Ill.
Galien Concrete Tile Co., Galien, Mich. (C. A. Roberts, Sec. & Treas.)
Gallagher, A. C., 2020 Voorhies Ave., Brooklyn, N. Y.
Gallagher, John J. (See Goodwin-Gallagher Sand & Gravel Corp.)
Gallagher, John P., 16 Hawthorne St., Watertown, Mass.
Gamboa & Domingo, Gran Via No. 13, Bilbao, Spain
Gannett, J. K., The Austin Co., 120 Broadway, New York City
Gardiner, Lion (See Lakewood Engineering Co.)
Gardner, Franc E., 3123 Bloomingdale Rd., Chicago, Ill.
Gardner, Guy O., Ash Grove Lime & Portland Cement Co., Chanute, Kans.
Gardner, W. McG., 313 Grosvenor Ave., Westmount, P. Q., Canada
Gardiner, W. H. (See Winston Bros. Co.,)
Cincero-Garita, Ricardo, C. E., Calle de Versovia 58, Mexico City, Mexico
Garnsey, Arnold Hugh, City Engineer, Town Hall, Sydney, Australia
Garrett, W. S., c/o Barclays Bank, Circus Place, London Wall, London, England
Garties, George, 2400 Gilbert Ave., Cincinnati, O.
Gaskill, P. D. (See Whitehall Cement Mfg. Co.)
Gaston, H. F., Pennsylvania Brick & Tile Co. Inc., Westmoreland Wharves, Philadelphia, Pa.
Gatzke, Chas. (See Des Plaines Concrete Products Co.)
Gauthier, P. G., c/o Beauharnois Construction Co., Box 50, Beauharnois, Que., Canada
Gautier, R. C., 103 Park Ave., New York, N. Y.
Gazaleh, Edouard, 42 Rue Sultan Abdel Aziz, Mazarita, Alexandrie, Egypt
Gazder, Mahamed Hashim, Lloyd Barrage & Canals Construction, Mirpurkhas, Sind, India
*Gay, George R., (See Santa Cruz Portland Cement Co.)
General Cement Products Co., Suite 930, Grant Bldg., Pittsburgh, Pa. (R. D. Campbell)
General Material Company, 605 Buder Bldg., St. Louis, Mo. (H. F. Thomson)
Gentry, J. W., Tenn. Dept. of Hwys. & Public Works, Lebanon, Tenn.
Geupel, Carl M., 1017 Hume Mansur Bldg., Indianapolis, Ind.
Geyer, J. George (See Federal Cement Tile Co.)
*__Giant Portland Cement Co.,__ Pennsylvania Bldg., Philadelphia, Pa. (Chas. F. Conn)

Gibbs, F. H., c/o Municipal Supplies Ltd., 1192 University St., Montreal, Que., Canada
Gibson, Jr., Chas. H., Tuskegee Inst., Ala.
Gibson, W. Herbert, Harrison Bldg., Philadelphia, Pa.
Giesecke, F. F. (See Texas Engineering Experiment Sta.)
Giesecke & Harris, 205 W. 7th St., Austin, Tex. (Munsey Wilson)
Gilbert, W. T. (See C. W. Blakeslee & Sons, Inc.)
Giles, Roy T., Atlas Lumnite Cement Co., 135 E. 42nd St., New York, N. Y.
Gilkey, Herbert J., 205 Engineering Bldg., No. 1, University of Colorado, Boulder, Colo.
Gill, Grayson, 307 Construction Industries Bldg., Dallas, Texas
Gillette, Harold S., 1913 Fort Worth National Bank Bldg., Fort Worth, Texas
Gillewicz, Zdzislaw, Nowogrodzka 25, Warszawa, Poland
Gilligan, William H., Terminal Development Co., 26 Exchange Place, Jersey City, N. J.
Gillon, Frank, 150 Davenport Ave., New Haven, Conn.
Gillon, Gerald Norton, 12 Manilla St., Dannevirki, New Zealand
Gilman, Charles, 50 Church St., New York, N. Y.
Gilmore, David P. (See University of Detroit Library)
Ginder, J. W., 439 Treasury Bldg., Washington, D. C.
Ginsberg, Frank I., H. O. Penn Co., 140th St. & E. River, New York,. N. Y.
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Glassett, Alfred T., Barney Ahlers Const. Corp., 110 W. 40th St., New York, N. Y.
Gleason, Kate, Beaufort, Sea Islands, S. C.
Gleason, W. A., Box 1878, Seattle, Wash.
Glen, Michael G., Division of Highways, San Bernardino, Calif.
***Glens Falls Portland Cement Co.,** Glens Falls, N. Y. (Geo. F. Bayle, Jr., Vice-Pres.)
* **Glens Falls Portland Cement Co.,** 205 Lower Warren St., Glens Falls, N. Y. (G. F. Bayle)
Glenn, B., Oregon State College, Corvallis, Oregon
Glover Company Inc., Geo. J., Whitney-Central Bldg., New Orleans, La.
Goddard, Calvin S., 69 Woodmont St., Portland, Me.
Goetz, John A., Mattoon, Ill.
Goldbeck, A. T. (See National Crushed Stone Assn.)
Golden Bay Cement Co., Ltd., The, P. O. Box 486, Wellington, New Zealand
Goldsmith Metal Lath Co., The, Chickering Ave. & B. & O., Cincinnati, Ohio (Louis I. Zagoren, Ch. Engr.)
Gomez-Perez, Francisco, Alfonso Herrera 56, Mexico City, Mexico
Gonnerman, H. F., Portland Cement Association Research Laboratory, 33 West Grand Ave., Chicago, Ill.
Gonzalez, Adriano, P. O. Box 1438, San Juan, Puerto Rico
Gonzalez, Carl E., P. O. Box 3591, Santurce, Porto Rico
Goode, R. A., The Building & Gen. Trading Co., Moshi, Tanganyika, E. Africa
Dooding, O. E. (See Standard Gravel Co.)
Goodman, E. M. (See City of Evanston)
Goodman, Hal W., Box 347, Portsmouth Ohio
***Goodwin-Gallagher Sand & Gravel Corp.,** 21 East 40th St., New York, N. Y. (John J. Gallagher)
Goodwin, L. A. (See Concrete Specialties Co.)
Gorman, Walter J. (See Sureseal Manufacturing Co.)
Gorsuch, V. D., Bessemer Cement Corp., Youngstown, Ohio
Goss, R. G., 13 Rainsford Road, Toronto 8, Ont., Canada
Gottlieb, R. D., 2000 Colquitt St., Houston, Tex.
Gottschalk, L. F., Columbus, Nebraska
Gottschalk, Otto, Buenos Aires, Arg., U. S. address 137 Smith St., Perth Amboy, N. J.
Gould Contracting Co., P. O. Box 56, 723 Argyle Ave., Nashville, Tenn. (C. B. Wilson)
Gould, Harley J., c/o Ferro-Concrete Constr. Co., 3rd and Elm Streets, Cincinnati, Ohio
Gow, Charles R., 38 Charles River Road, Cambridge, Mass.
Gowdy, Roy C., 457 Railway Exchange Bldg., Denver, Colo.
Gräf, Otto, Spittler-strasse 18c, Stuttgart, Germany
Gram, Lewis M., 227 West Engineering Bldg. University of Michigan, Ann Arbor, Mich.
Granite Rock Co., Watsonville, Calif. (R. E. Fowle)
Grant, J. R., 1202 Bekins Bldg., Vancouver, B. C., Canada
Grasselli Chemical Co. Inc., The, Guardian Bldg., Cleveland, Ohio (R. F. Remler)
Grassi & Co., P., 1945 San Bruno Ave., San Francisco, Calif. (P. Grassi)
Grassi, P. (See P. Grassi & Co.)
Gravell, Inc., Wm. H., N. E. Cor. 15th & Locust St., Philadelphia, Pa. (Wm. H. Gravell, Pres.)
Gravell, Wm. H., (See Wm. H. Gravell Inc.)
Gray Concrete Co., Thomasville, N. C. (F. B. Gray, Secy. & Treas.)
Gray Construction Co., John V., 330 Bay St., Toronto, 2, Ont., Canada (F. S. Milligan)
Gray, F. B. (See Gray Concrete Co.)
Gray, Howard A., 862 Park Sq. Bldg, Boston, Mass.
Gray, T. Campbell, The Concrete Association of India, 4/35, Khaleeli Mansions, Mount Rd., Madras, India
Gray, William J., P. O. Box 292, Springfield, Mo.
Great Lakes Concrete Pipe Co., Inc., 973 Ellicott Square, Buffalo, N. Y. (J. G. Miller, Treas.)
Greater Winnipeg Water District, Civic Offices, Winnipeg, Manitoba. (W. M. Scott, Chairman of Commissioners)
Green, Howard R., Bever Bldg., Cedar Rapids, Ia.
Greenfield, Inc., Arthur, 1 Union Sq., West, New York, N. Y. (Arthur Greenfield)
Greenfield, Arthur (See Arthur Greenfield, Inc.)
Greenfield, Frank L. (See Gulick-Henderson Co.)
Greenman, Russell S., 10 Steuben St., Albany, N. Y.
Gregson, Harry St. John-Russell-de-Lys-, c/o Sir J. Jackson Ltd., H. M. Naval Base, Singapore, Straits Settlements
Greiner, J. E., Lexington Bldg., Baltimore, Md.
Grid Flat Slab Corp., 761 Dudley St., Boston, Mass. (Edwin F. Allbright)
Griffith, Norman (See Nonporite Pty. Ltd.)
Grisi, Adolfo P., Calle 5 No. 1380, LaPlata, Argentina
Grone, Edwin Arthur, 3044 You St., Lincoln, Nebr.

Grothe, Wm. F., 308 East "B" St., Hutchinson, Kans.
Groupement Prof. des Fabricants de Cement Portland Artificial de Belgium, 45, Boulevard Du Regent, Brussels, Belgium (G. Deroover)
Grun, Dr. Richard, Director, Forschungsinstitut der Huttenzement Industrie, Dusseldorf, Germany
Grundt, Odd, Torvet 9, Oslo, Norway
Grutsch, Frank J. (See St. Louis Concrete Products Mfg. Co.)
Guaranteed Concrete Co., 210 Builders Exchange, St. Paul, Minn. (A. R. Shiely, V. P.)
Guarantee Construction Co., 17 Battery Place, New York City, N. Y. (Edward Burns, Vice-Pres.)
Guarello, Angel (See Inspeccion de Caminas)
Guest, Walter C., C. M. Guest & Son, Anderson, S. C.
Gulf Concrete Pipe Co., P. O. Box 1765, Houston, Texas (H. E. Johnsen, Vice-Pres.)
Gulick-Henderson Co., 19 West 44th St., New York City (Frank L. Greenfield)
Gundersen, August (See A/S Hoyer-Ellefsen)
Gunn, Charles M., Gunn, Carle & Co., Ltd., 444 Market St., San Francisco, Calif.
Gutierrez-Salinas, Jorge B., Santa Fe 951, Buenos Aires, Argentina
Hackett, Thos. P., The Cincinnati Cement Products Co., 2022 Dana Ave., Cincinnati, Ohio
Hadden, Gavin, 607 Fifth Ave., New York, N. Y.
Haddow, Jr. Hugh, Milville, N. J.
Hadley, H. M., 518 Exchange Bldg., Seattle, Wash.
Haegermann, Dr., Donhoffstr 38, Berlin, Karlshorst, Germany
Haegert, Lee V., 2240 W. 8th St., Topeka, Kans.
Haenke, Martin William, Ipswich, Queensland, Australia
Haertlein, Albert, Harvard Engineering School, Pierce Hall, Cambridge, Mass.
Hegener, Arthur (See Whitacre Engineering Co.)
Hagerty, L. D. (See Cunard Lang Concrete Co.)
Haggart, C. N., 2709 Murray Ave., Morrowfield Apts., Pittsburgh, Pa.
Hagy, Ernest A., Box 588, Cincinnati, Ohio
Hahn, Harold S., 2018 E. Greenwich Ave., Milwaukee, Wisc.
Hall, Quincy A., 212 Metropolitan Bank Bldg., St. Paul, Minn.
Hall Construction Co., 500 Board of Trade Bldg., Indianapolis, Ind.
Hallahan, John P., 3116 Market St., Philadelphia, Pa.
Halliburton Oil Well Cementing Co., Duncan, Oklahoma (Hayden Roberts, Chemist)
Halloran, Lt. P. J., (Cec) U. S. N., 200 25th St., Jackson Heights, L. I., N. Y.
Hambly, Percy Noel, Industrial Constrs., Ltd., 54 Victoria St., London, S. W. I., England
Hamburger, A. (See Ferro Building Products Co., Inc.)
Hamel, Edouard (City of Quebec)
Hamilton, Chas. T., 2044 44th Ave. W., Vancouver, B. C., Canada
Hamilton Gravel Co., The, North Third St., Hamilton, Ohio (W. P. Watson, Sec.-Treas.)
Hamilton & Weeber, 624 Grand Rapids Nat'l Bank Bldg., Grand Rapids, Mich. (E. R. Weeber)
*Hamlin, H. P. (See Raymond Concrete Pile Co.)
Hammerschmidt, Martin (See Elmhurst-Chicago Stone Co.)
Hammill, Harold B., 42 Portsmouth Road, Piedmont, Oakland, Calif.
Hammond, Inc., Ross B., 1312 Public Service Bldg., Portland, Ore. (Ross B. Hammond)
Hammond, Ross B. (See Ross B. Hammond, Inc.)
Hand, Mrs. Elsie D. (See Oklahoma A. & M. College Library)
Hand, George T., Chief Engr., Lehigh Valley Railroad, 143 Liberty St., New York, N. Y.
Hands, Stanley M., 218 Johnson Co., Bank Bldg., Iowa City, Ia.
Hanks, Inc., Abbott A., 624 Sacramento St., San Francisco, Calif. (Theo. P. Dresser, Jr., Chief Engr.)
Hanley, Hunter W. (See Euphrat-Hanley)
Hanley, J. T. (See American System of Reinforcing)
Hanna, Frank W., 512 16th St., Oakland, Calif.
Hanna, H. B., 313 Elias Ave., Peterboro, Ont., Canada
Hanna, W. C. (See California Portland Cement Co.)
Hannaford, H. Eldridge, 1024 Dixie Terminal Bldg., Cincinnati, Ohio
Hannaford, W. S. P. (See W. J. Hynes, Ltd.)
Hannan, Frank J. (See Richard Taylor & Co.)
Hanneman Bros., 7841 South St., Detroit, Mich.
Hannock, Charles F., County Engineer's Office, Miami, Fla.
Hansard, Orren H., 174 Third Ave., N., Nashville, Tenn.
Hansen, Ralph, Central Y. M. C. A., Cleveland, Ohio
Harboe, Prof. Ed., Consulting Engr., Trondhjem, Norway
Harboe, Helge, Scott & Urner, Caixa Postal, 1863, Rio de Janeiro, Brazil
Harber, G. Peyton, Albany, Oregon
Hardesty, J. M. (See Bell Telephone Laboratories)
Harding, Carroll R., 65 Market St., San Francisco, Calif.
Harding, E. C., Ferro-Concrete Construction Co., 3rd & Elm Sts., Cincinnati, Ohio
Hardy, A. W., 303 N. Washington, Magnolia, Ark.
Hargen, Stanley, 133 Rutland Road, Brooklyn, N. Y.
Haring, Ellis E., 1744 Bellaire St., Denver, Colo.
Harkness, E. B. (See Illinois Steel Co.)
Harkness, London & Hertzberg, 17 Queen St., E., Toronto 2, Ont., Canada
Harkin, J. B. (See Canadian National Parks Branch)
Harm, George O., P. O. Box 727, Warren, Ohio
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Harris, Robert Olin, 956 Waverly Way, N. E., Atlanta, Ga.
Harris, Wallace R., Knickerbocker Hotel, Milwaukee, Wis.
Harrison, Allrich S. (See Minwax Company, Inc.)
Harrison, Dr. H. C. (See Westinghouse Electric & Mfg. Co.)
Harrison, Merritt, Harrison & Turnock, 1001 Architects Bldg., Indianapolis, Ind.
Hart, W. E., 33 W. Grand, Ave., Chicago, Ill.
Harter-Marblecrete Stone Co., 1608-14 W. Main St., Oklahoma City, Okla. (B. D. Harter)
Harter, B. D. (See Harter-Marblecrete Stone Co.)
Hartford, City of, Dept. of Buildings, 550 Main St., Hartford, Conn. (Philip A. Mason, Bldg. Supervisor)

Hartless, T. B. (See Froeling & Robertson)
Hartley, D. R. C., c/o Messrs. Killick Nixon & Co., Cement Department, Post Box No. 109, Bombay, India
Harty, John J., 25 Huntington Ave., Boston, Mass.
Harza, L. F., Cons. Engr., President, Harza Engineering Co., 20 N. Wacker Drive, Chicago, Ill.
Hatch, L. A. (See Minn. Mining & Mfg. Co.)
Hatch, Margaret (See Standard Oil Co. Library)
*Hatch, Wm. M. (See Peerless Portland Cement Co.)
Hatt, William K., Purdue University, Lafayette, Ind.
*Haubert, E. P. (See Lawrence Portland Cement Co.)
Haupt, Chas. E. (See Standard Oil Development Co.)
Hausman Steel Co., The, P. O. Box 416, Toledo, Ohio
*Havemeyer, J. F. (See Concrete Steel Co.)
Hawaiian Contracting Co., Ltd., 854 Kaahumanu St., Honolulu, T. H. (H. P. Benson, Mgr.)
Hawley, Geo. W., 1100 N. Street, Sacramento, Calif.
Hay, Alan K. (See Ottawa Suburban Roads Commission)
Hay, William Wren, c/o Cox & Jordan, 40 Wall St., New York, N. Y.
Hayden, J. M., Calif. Highway Commission, San Simeon, Calif.
Hayden, R., 420 Lexington Ave., New York N. Y.
Haydite Co., The, 403 Mutual Bldg., Kansas City, Mo. (Dan F. Servey)
Haydock, Winters (See Department of City Transit, City of Pittsburgh)
Hayes Engineering Corp., J. E., 49 Taku Road, Tientsin, China (J. K. Davison)
Hayes, Walter W., c/o Turner Constr. Co., 90 Dover Road, Wellesley, Mass.
Hayley, Harry, Hurdman Road, Ottawa, Canada
Hayner, J. L., 30 Parkview Ave., Pittsburgh, Pa.
Haynes, S. B., 511 Myrick Bldg., Lubbock, Texas
Headman, Sasha, Tucson Gas, Electric Light & Power Co., Tucson, Ariz.
Healy, Ralph F. (See Igoe Brothers)
Hebblewhite, W. R. (See Standards Association of Australia)
Hebold, Denis O., 217 West Sparks St., Philadelphia, Pa.
Hedderich, H. F., c/o Westinghouse Elec. & Mfg. Co., 3850 Bingham Ave., St. Louis, Mo.
Heidenreich, E. Lee, 618 Kansas City Life Bldg., Kansas City, Mo.
Heidenreich, Jr., E. Lee, 252 Water St., Newburgh, N. Y.
Heilbronner, L. C., Caixa Postal 2947, Sao Paulo, Brazil
Henderson, H. B. (See Corwin & Co., Ltd.)
Hendrickx, Jean, 125 Quai de Valmy, Paris, Xeme, France
Henny, D. C., Spalding Bldg., Portland, Oregon
Hepburn, William, 2 Primrose St., White Plains, N. Y.
***Hercules Cement Corp.,** Nazareth, Pa.
Hernandez, Inc., Clinton N., 208-210 McLean Ave., Yonkers, N. Y. (Clinton N. Hernandez)
Hernandez, Clinton N. (See Clinton N. Hernandez, Inc.)
Herron, Co., The James H., 1360 W. Third St., Cleveland, Ohio (James H. Herron)
Herron, James H. (See The James H. Herron Co.)
Hersey Co., Ltd., Milton, 980 St. Antoine St., Montreal, Que., Canada (M. F. MacNaughton)
Hersum, LeRoy M., 6 Beacon St., Boston, Mass.
Hertz, A. L., Box 38, Balbao, Canal Zone
Hertz, H. P., Routledge & Hertz, Archts. & Engrs., 204-5 Ex. Nat'l Bank Bldg., Hutchinson, Kan.
Hewes, George C., 403 Chestnut St., Haddonfield, N. J.
Hews, W. P. (See Yakima Cement Products Co.)
Hibbs, Manton E., Upsal Gardens, Germantown, Philadelphia, Pa.
Hickerson, T. F., Chapel Hill, N. C.
Hidalgo, Rafael A., Casilla 979, Panama, Republic de Panama
Higgins, N. B. (See Pennsylvania Water & Power Co.)
Hilker Supply Co., 16th & State St., Granite City, Illinois (E. W. Hilker, Pres.)
Hilker, E. W. (See Hilker Supply Co.)
Hill, Louis C. (See Quinton Code & Hill)
Hill & Taylor, A. M. P. Buildings, Cairns, North Queensland, Australia (A. J.Taylor)
Hill, W. N., 519 Loucko Ave., Peoria, Ill.
Hilts, H. E., 123 So. Broad Street, Philadelphia, Pa.
*Hilts, H. E. (See Lone Star Cement Co.)
Hinchman, T. H., Smith, Hinchman & Grylls, Marquette Bldg., Detroit, Mich.
Hinder, R. B., Kandos Cement Co., Ltd., 66 Pitt St., Sydney, N. S. W., Australia
Hindman, W. S., 172 E. North Broadway, Columbus, Ohio
Hipp, George E. (See Buffalo Litholite Co.)
Hipwell, Howard S., 1425 City Hall Annex, Philadelphia, Pa.
Hirschberg, Walter P., 218 Stephenson Bldg., Milwaukee, Wisc.
*Hirsh, A. M. (See Lock Joint Pipe Co.)
Hirth Concrete Breaking Co., 3851 No. Hubbard St., Milwaukee, Wis. (G. W. Hirth)
Hirth, G. W. (See Hirth Concrete Breaking Co.)
Hislop, W. A., Box 128, Leeton, N. S. W., Australia
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Hodge, Ernest R., Education Office, Wanganui, N. Z.
Hodges Construction Co., The, Chamber of Commerce Bldg., Cincinnati, Ohio (J. E. Hodges)
Hodges, J. E. (See The Hodges Construction Co.)
Hoeffer & Co., 308 W. Washington St., Chicago, Ill. (Alexander C. Warren)
Hoerr, Philipp J., 1104 Frye Ave., Peoria, Ill.
Hoffman, A. E., Gen. Mgr., Clinton Motors Corp., Reading, Pa.
Hoffman Co., F., 901 Wood St., Wilkinsburg, Pa. (Wm. F. Hoffman)
Hoffman, L. R., 1143 Buckingham Rd., Grosse Pointe Park, Mich.
Hoffman, R. B., 14 Del Monte Ave., Monterey, Calif.
Hoffman, Wm. F. (See F. Hoffman Co.)
Hoke, Arnold, Cementon, Pa.
Holabird & Root, 333 N. Michigan Ave., Chicago, Ill. (E. A. Renwick)
Holdsworth, Harry (See Holdsworth Patents Inc.)
Holdsworth Patents Inc., 250 W. 57th St., New York, N. Y. (Harry Holdsworth)
Holleran, L. G. (See Westchester County Park Commission)
Hollister, S. C., Dept. of Structural Engineering, Purdue University, Lafayette, Ind.

Hollywood Concrete Pipe Co., P. O. Box 2057, DeSoto Station, Memphis, Tenn. (O. H. Miller, Pres.)
Hollywood Building Block Co., No. Plymouth St., Allentown, Pa. (Paul R. Klotz)
Holm, W. M., Springer Transfer Co., Inc., P. O. Box 572, Albuquerque, New Mexico
Holmes, Ltd., A. R., 11 Castle Frank Cres., Toronto, Canada (A. R. Holmes, Pres.)
Holmes, A. R. (See A. R. Holmes, Ltd.)
Holmes, Francis, 208-12 Tranaki St., Wellington, New Zealand
Holst, Helge (See Teknisk Bibliotek)
Holter, A., Brevik, Norway
Holton, Jr., Philip J., 389 Charles St., Providence, R. I.
Holz, Herman A., 167 East 33rd St., New York, N. Y.
Holtz & Co., Inc., R. B., Grand Central Terminal, New York, N. Y. (R. B. Holtz)
Holtz, R. B. (See R. B. Holtz & Co., Inc.)
Honig, B. A. (See Vigorose Cement Industrie, N. V.)
Hooks, James H., Alberene Stone Co., 153 West 23rd St., New York, N. Y.
Hool, George A., c/o The Shelton, New York, N. Y.
Hopton, H. V., 306 Cowan St., Nashville, Tenn.
Horn, A. E., Horn Building, Long Island City, N. Y.
*Horner, A. W. (See Kosmos Portland Cement Co.)
Horner, J. M., 250 Conway St., Carlisle, Pa.
Horner & Wyatt, 470 Board of Trade Bldg., Kansas City, Mo. (Roy M. Wyatt)
Horner, Wesley W., 300 City Hall, St. Louis, Mo.
Horst Co., Henry W., 1505 Race St., Philadelphia, Pa. (A. E. Horst)
Horst, A. E. (See Henry W. Horst Company)
Hoskins, C. H., Southern Portland Cement Ltd., Kembla Bldg., Margaret St., Sydney, N. S. W., Australia
Houk, Howard H., 1209 S. Court St., Montgomery Ala.
Houston, City of, City Hall, Houston, Tex. (H. L. Show, City Engr.)
Hovde, A. C. (See Capital Avenue Cement Works)
Howard, S. B., 26 Orchard St., Biddeford Maine
Howe, C. D., The Whelan Bldg., Port Arthur, Ont., Canada
Howe, Henry L. (See City of Rochester, N. Y. Director of Design and Construction)
Howe, H. N., 76 Porter Bldg., Memphis, Tenn.
Howell, C. H., c/o The J. G. White Engineering Corp., 2 A. San Juan de Letran 24, Mexico, D. F., Mexico
Howland, M. R., 2500 Peralta St., Oakland, Calif.
Hoyer-Ellefsen, A/S, P. O. Box 39, Oslo, Norway (August Gundersen, Chief Engr.)
Hoyt, C. R., St. George Hotel, Brooklyn, N. Y.
Hoyt, Glenn T., 80 Whitman Ave., Melrose, Mass.
Hubbard, Fred, 1200 City Park Bldg., Youngstown, Ohio
Hubbell-Hartgering & Roth, 2640 Buhl Bldg., Detroit, Mich. (Clarence W. Hubbell
Hubbell, Clarence W. (See Hubbell-Hartgering & Roth)
Huber, Lee, 1828 I St., Bedford, Ind.
Huber, Walter L., 1 Montgomery St., San Francisco, Calif.
Hud-Cin Building Products Inc., Van Keurer Ave., Jersey City, N. J. (W. H. Blackwood)
Hudson Block & Supply Co., Inc., 5274 Hudson Blvd., North Bergen, N. J. (W. J. Dicker, Gen Mgr.)
Hudson, W. G. (See Portland Cement Assn.)
Hughes, C. A., Room 203 Experimental Engineering Bldg., University of Minnesota, Minneapolis, Minn.
*Hughes, E. E. (See Franklin Steel Works)
Hughes, R. G., 152 Market St., Paterson, N. J.
Hull, Herbert F., Box 2037, Seattle, Wash.
*Hulsart, C. R. (See Universal Atlas Cement Co.)
Humberd, John L. (See Ready Mixed Concrete Co.)
Humboldt Mfg. Co., 2014 Nebraska Ave., Chicago, Ill (Louis H. Martens, Pres.)
Hume, Albert S., Cangallo 439, Buenos Aires, Argentina
Hume Pipe Co. (South Africa) Ltd., The, P. O. Box 204, Germiston, Transvaal, South Africa (Kenneth Ray)
Hume, Walter Reginald, Hume Steel Ltd., Kinnear House, King & Little Collins Sts., Melbourne, Australia
Humphrey, D. S., Euclid Beach Park, Cleveland, Ohio
Hunt & Co., Robert W., 53 Park Place, New York N. Y. (J. F. Davis)
Hunt & Co., Ltd., Robert W., 901 McGill Bldg., Montreal, Canada (F. O. Farey)
Hunt, Thomas D., 536 County City Bldg., Seattle, Wash.
Hunt, W., P. O. Box 340, Townsville, Queensland, Australia
Hunter, Harry B., Roof U. & P. Bldg., Memphis, Tenn.
Hunter, Russell H. (See Starrett Brothers & Eken, Inc.)
Huntington, W. C., 201 Engineering Hall, University of Illinois, Urbana, Ill.
Hurlburt, R. W., 52 York St., Toronto, Ont., Can.
Hurley, John J., 41 Bonair St., Somerville, Mass.
***Huron Portland Cement Co.,** Ford Building, Detroit, Mich. (J. W. Kennedy)
Huser, Dr. Ing. Alfred, Obercassel-Siegkreis, Germany
Hussey, Jack W., c/o Universal Atlas Cement Co., 1012 Baltimore Ave., Kansas City, Mo.
Hustad Company, The, 521 Sexton Bldg., Minneapolis, Minn. (A. P. Hustad, Pres.)
Hustad, A. P. (See The Hustad Company)
Hutchinson, G. W. (See Riverton Lime Co.)
Hutchinson, Jr., Hardye, 1450 Harrison St., Oakland, Calif.
Hutter, George F. (See Hutter Construction Co.)
Hutter Construction Co., 128 Western Ave., Fond du Lac, Wis. (George F. Hutter)
Hutton, S. E., 1542 17th Ave. No., Seattle, Wash.
Huxhold, P. F. (See Concrete Bldg. Units Co.)
Hyde, George A., c/o Chicago Stone Conduit Co., 3595 N. California Ave., Chicago, Ill.
Hydraulic Press Brick Co., South Park, Ohio
Hydro-Electric Power Comm., 190 University Ave., Toronto, Ont.
Hydroprest Products, Ltd., North Lincoln Rd., Scunthorpe, Lincs, England (A. Ogilvie)
Hynes, Ltd., W. J., 29 Birch Ave., Toronto, 5, Canada (W. S. P. Hannaford)
Idaho, Dept. of Public Works, Box 1467, Boise, Idaho (J. D. Wood)
Ideal Concrete Co., Inc., Joliet. Ill. (J. G. Sengenberger)
Ideal Concrete Co. Inc., 455 Rowell Ave., Joliet, Ill. (John G. Sengenberger)
Igoe Brothers, Ave. A & Pointer St., Newark, N. J. (Ralph F. Healy)

Illinois Division of Highways, Bureau of Materials, 100 E. Washington St., Springfield, Ill.
Illinois Steel Co., 208 S. LaSalle St., Chicago, Ill (E. B. Harkness)
Imlay, R. G., 704 East End Ave., Pittsburgh, Pa.
Immel Construction Co., 198 N. Main St., Fond du Lac, Wisc. (H. M. Mabie, Jr.)
Imprese Idrauliche Elettriche del Tirse, Sardegna, Cagliari, Italy
Independent Concrete Pipe Co., 201 N. West Street, Indianapolis, Ind. (R. D. Robertson)
Independent Concrete Pipe Co., Ltd., 198 Riddell St., Woodstock, Ont., Canada (James Carnwath)
Indian Hume Pipe Co. Ltd., The, Phoenix Building, Ballard Est., Bombay, India
***Indiana Portland Cement Co.,** Indianapolis Ind. (George E. Pierson, Mgr.)
Industrial Engineering Co., 30 Church St., New York City, N. Y. (D. Traver Miller)
Infante, Percival M., New York Central R. R. Lines, 466 Lexington Ave., New York, N. Y.
Ingberg, S. H., Bureau of Standards, Washington, D. C.
Ingemanson, Thure W., 5944 W. Erie St., Austin Sta., Chicago, Ill.
***Inland Steel Co.,** First Nat'l Bank Bldg., Chicago, Ill. (A. C. Roeth)
***Insley Mfg. Co.,** Indianapolis, Ind. (A. C. Rasmussen)
Integral Waterproofing Co., 300 Water St., New York, N. Y. (Frank J. Phelan)
International Cement Corp., 342 Madison Ave., New York, N. Y. (Howard Rhode)
***International Cement Corp.,** 342 Madison Ave., New York, N. Y.
***International Cement Corp.,** 342 Madison Ave., New York, N. Y.
Iowa Concrete Crib & Silo Co., 820 S. W. Ninth St., Des Moines, Iowa (H. E. Kilmer, Gen. Mgr.)
Iron City Sand & Gravel Co., Cumberland, Maryland
Irwin, Orlando W., 401 Alameda Ave., Youngstown, Ohio
Israil, Mohammad, c/o Maulana Maulavi Saiyed Mohd Abdul Hafeez Sahib, Galee Maulavi Abdul Hakin, Phatak Habash Khan, Delhi, India
"Italcementi", Fabriche Riunite Cemento, Bergamo, Italy (Carlo Vigliani)
Jack, Erick D., Waimea County Council, Nelson, New Zealand
Jack, Eugene C., P. O. Box 893, Bremerton, Wash.
Jack, W. A. (See Maine Concrete Co.)
Jacobson, C. W., 104 Greenhill Parkway, Worcester, Mass.
Jackson, F. H., U. S. Bureau of Public Roads, Washington, D. C.
Jackson-Lewis Co., The, 1114 Federal Bldg., Toronto, Ont., Canada (C. Blake Jackson, Pres.)
Jackson, C. Blake (See The Jackson-Lewis Co.)
Jackson & Moreland, 31 St. James Ave. Boston, Mass. (E. H. Cameron)
Jacoby, H. S., Hanna Bldg., Cleveland, O.
Jakhelln, Harald (See Christiania Portland Cementfabrik)
James, W. H., "Newlyn", 4th Ave., Joslin, Adelaide, South Australia
James, Proctor & Redfern, Ltd., 36 Toronto, St., Toronto, Ont. (W. D. Proctor)
James, William, 1609 Franklin Ave., St. Louis, Mo.
Jameson, R. O., 1005 Southwestern Life Bldg., Dallas, Texas
Jamestown Block & Tile Co., P. O. Box 712 Jamestown, N. Y., (D. A. McClure, Treas.)
Jeffers, Paul E., 816 West 5th St., Los Angeles, Calif.
Jeffrey, W. McKenzie, P. O. Box 1734, Auckland, N. Z.
Jellick, J. E., Calaveras Cement Company, 315 Montgomery St., San Francisco, Calif.
Jennings, Wm. A., Box 322, Osborn, Ohio
Jewett, John Y., Administration Bldg., Balboa Park, San Diego, Cal.
Jimenez L., Cesar, c/o Comision N. De Irrigacion, Estacion La Cruz, Chihuahua, Mexico
Joffe & Co., A. S., P. O. Box 4147, Johannesburg, S. Africa (A. S. Joffe))
Joffe, A. S. (See A. S. Joffe & Co.)
Joglekar, M. V., c/o Tata Constr. Co. Ltd., Phoenix Bldg., Ballard Estate, Fort, Bombay 10, India
Johns, H. S., Hillyard St., Hamilton, Ont., Canada
***Johns-Manville Corp.,** 292 Madison Ave., New York, N. Y. (Geo. A. Smith, Engr.)
Johnsen, E., c/o Francisco & Jacobus, 511 Fifth Avenue, New York, N. Y.
Johnson, A. (See Central Concrete Mixing Corp.)
Johnson, A. F., 1411 Foshay Tower, Minneapolis, Minn.
Johnson, Axel H., 1192 Gladys Ave., Cleveland, Ohio
Johnson, A. N. (See University of Maryland)
Johnson, A. W., 2037 Morse Ave., North Town Sta., Chicago, Ill.
Johnson C. H. (See Benson Concrete Co.)
Johnson Co., C. S., 1002 North Market St., Champaign, Ill (Chas. S. Johnson)
Johnson, Chas. S. (See C. S. Johnson Co.)
Johnson, H. E. (See Gulf Concrete Pipe Co.)
Johnston, Hugh H. (See South Dakota Concrete Products Co.)
Johnston, Harold S. (See The Nova Scotia Power Comm.)
*Johnston, J. W. (See Lone Star Cement Co.)
Johnson, Lewis J., Harvard University, Cambridge, Mass.
Johnson, L. E. (See The Finishing Lime Assn. of Ohio)
Johnson, Otis Floyd, 6016 Ellis Ave., Chicago, Ill.
Johnson, O. K., 1000 Franklin Ave., Waco, Texas
Johnson, Robert C., 19 W. Division St., Fond du Lac, Wisc.
Johnson, R. W. (See Concrete Reinforcing Steel Institute)
Johnston, Robert Smith, Bayside Blvd., Bayside, N. Y.
Johnson, T. H., 319 Iowa Bldg., Sioux City, Iowa
Johnson, Virgil L., 29 W. Upsal St., Germantown, Philadelphia, Pa.
Johnston, W. J., P. O. Box 1417, St. John, N. B., Canada
Johnson, W. P., c/o Meyers & Johnson, Commerce Bldg., Erie, Pa.
Johnson, Wm. R., Box 588, Cincinnati, Ohio
Jones, Bevan, 101 Park Ave., New York, N. Y.
Jones-Hettelsater Constr. Co., 707 Mutual Bldg., 13th & Oak St., Kansas City, Mo. (Edmund Wilkes)
Jones & Laughlin Steel Corp., South Side Works, Pittsburgh, Pa. (J. B. Carlock)
Jones, R. C., No. 233 Balboa Heights, Canal Zone
Jones, Walter E. (See Federal Concrete Co.)
Jones, William A., 148 Mahanoy Ave., Mahanoy City, Pa.
Jones, William M., 517 E. 28th St., Paterson, N. J.

Jorgensen, Lars, 1201 Hobart Bldg., San Francisco, Calif.
Jorss, Chris. A., P. O. Box 73, Maryborough, Queensland, Australia
Josephs, Arthur C., 300 E. Michigan St., Duluth, Minn.
Jourdan Concrete Pipe Co., Box 914, Fresno, Calif. (H. W. Chutter)
Kahler, G. A. (See Prairie Concrete Products Co.)
Kahn, Albert, Marquette Bldg., Detroit, Mich.
*Kahn, Julius (See Truscon Steel Co.)
Kaiser, B. J., 801 Keystone Bldg., Pittsburgh, Pa.
Kaiser Paving Co., 1522 Latham Sq. Bldg., Oakland, Calif. (T. M. Price)
Kalinka, J. E., c/o Roberts Schaefer Co., Wrigley Bldg., Chicago, Ill.
Kalman Floor Co., 410 N. Michigan Ave., Chicago, Ill. (C. E. Cooke)
***Kalman Steel Co.,** Wrigley Bldg., Chicago, Ill.
Kamat, V. A., Shahpoor House, 114 Cawasji Patel St., Fort, Bombay, India
***Kansas Portland Cement Co., The,** Federal Reserve Bank Bldg., Kansas City, Mo. (J. A. Lehaney, Vice-Pres.)
Kao, P. K., Chuny-Hwa Book Co., Mukden, China
Kaufman, David M., 80 Van Cortlandt Park South, New York, N. Y.
Kaye, Lister Lister, Southam Works, Rugby, England
Kealy, J. T. (See Binney & Smith Co.)
Kearney, James C., 1758 Penobscot Bldg., Detroit, Mich.
Kearney, Stephen (See Cinder Concrete Units Corp.)
Keatinge, Paul F., 208 S. LaSalle St., Chicago, Ill.
Keesee, Gerald B., P. O. Box 118, Brownwood, Texas
Keily, J. V. (See R. I. State Bd. of Public Roads)
Kelch, Norman (See Clay Products Institute of California)
Kellam, Fred, State Highway Commission, State House Annex, Indianapolis, Ind.
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Kelley, Frederick W., 40 Howard St., Albany N. Y.
Kelley, J. W., 5328—2nd Ave. So., Minneapolis, Minn.
Kemmer, A. E., Lafayette, Indiana
*Kennedy, J. W. (See Huron Portland Cement Co.)
Kennedy, W. A. (See Reichard-Coulston, Inc.)
Kennoy, John S., 220 Golden Gate Ave., San Francisco, Calif.
Kentucky State Highway Dept., Frankfort, Ky. (J. S. Watkins)
Kerr, Frank O. (See College of Engineering, University of Porto Rico)
Kerr, G. M. (See Core Joint Concrete Pipe Co., Inc.)
Kerr, George W., State Highway Dept., 307 Trust Bldg., Newark, Ohio
Kerr, Linton, 49 Federal St., Boston, Mass.
Kerwin, P. A. (See Dept. of Street Railways, Detroit, Mich.)
Ketchum, M. S., Dean of College of Engineering, University of Illinois, Urbana, Ill.
*Kichline, H. F. (See North American Cement Corp.)
Kienstra Bros. Fuel & Supply Co., Wood River, Ill. (Frank T. Kienstra)
Kienstra, Frank T. (See Kienstra Bros. Fuel & Supply Co.)
Kikkeri, S. A., B. E., c/o Treasury Officer, T. Narsipur, Mysore, S. India
Killian, M. B., c/o R. W. Briggs & Co. Victoria, Texas
Kilmer, H. E. (See Iowa Concrete Crib & Silo Co.)
Kimpton, A. C., Casebourne & Co., Ltd., "Pioneer" Cement Works, Haverton Hill, nr. Middbr., England
Kindle, W. H., 798 Broadway, Newark, N. J.
King, A. W. (See Consolidated Cement Corp.)
King, Thomas H., 716 Electric Bldg., San Diego, Calif.
King, W. E., Builders Exchange Bldg., St. Paul, Minn.
Kinneman, W. P., 140 Cedar St., New York, N. Y.
Kinney, Wm. M., 33 W. Grand Ave., Chicago, Ill.
Kirchhof Constr. Co., F. J., 700 Lawrence St., Denver, Colo. (F. J. Kirchhof)
Kirchhof, F. J. (See F. J. Kirchhof Constr. Co.)
Kirk, Joe Neil, 115 Drexel Ave., Lansdowne, Pa.
Kirwan, Kenneth K., 923 Munsey Bldg., Baltimore, Md.
Kitson, C. O. (See Blue Ridge Talc. Co., Inc.)
Kivett, Edward H., Louisiana Highway Commission, Baton Rouge, La.
Klank, Willy, c/o Struck. Galvanistr. 6, Berlin-Charlottenburg, Germany
Klassy, K. H., Stockton, Ill.
Klein, W. H., Pennsylvania-Dixie Cement Co., 1010 James Bldg., Chattanooga, Tenn.
Kleinlogel, Prof. Adolf, Roquetteweg 33, Darmstadt, Germany (Hessen)
Kleinsteuber, M. E. (See Cut Art Stone Co.)
Kling, E. E. (See Youngstown Burial Vault Co.)
Klingensmith, F. (See American Hume Concrete Pipe Co.)
Klinger, W. A., Warnock Bldg., Sioux City, Iowa
Klock, Morgan B., 25 E. Jefferson Road, Pittsford, N. Y.
Klotz, Paul R. (See Hollywood Building Block Co.)
Kneeland, H. D. (See Calif. Stucco Products Co.)
Kneeshaw, F. P., Gen. Mgr., G. P. O. Box 2613 EE, 66 Pitt St., Sydney, N. S. W., Australia
Kneisel, Dr. Rudolf (See Verein Deutscher Portland)
Knight, W. J., Pres., W. J. Knight & Co., 903 Warnerright Bldg., St. Louis, Mo.
Knopel, Herbert J., Warner Company, 1518 Walnut St., Philadelphia, Pa.
Knox, Jean H., 1142 N. Edgefield Blvd., Dallas, Texas
***Koehring Company,** 31st & Concordia Ave., Milwaukee, Wis. (Philip Koehring)
***Koehring Company,** 31st & Concordia Ave., Milwaukee, Wis. (E. H. Lichtenberg)
Koehring, Philip (See Koehring Company)
Koenig Coal & Supply Co., 1480 Gratiot Ave., Detroit, Mich. (A. J. O'Connor)
Koepke Bros. Const. Co., 201 E. College Ave., Appleton, Wisc. (G. A. Koepke)
Koepke, G. A. (See Koepke Bros Const. Co.)
Koerner, Carl A., 1601 Syndicate Trust Bldg., St. Louis, Mo.
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Kohli, T. N., Dhakpuri, Alwar City (Rajputana) India
Kolinski, M. C., Pres., Kolinski Ready Mixed Concrete Co., 127 S. 11th St., Milwaukee, Wisc.
Komuro, Mangoro, Iwaki Cement Co. Ltd., Yotsukuracho, Fukushimaken, Japan
Kondow, Yasuo, Kyoto Imperial University, Kyoto, Japan
***Kosmos Portland Cement Co.,** 1430 Starks Bldg., Louisville, Ky. (A. W. Horner)

Krabbe, A. F. (See The Olympic Portland Cement Co. Ltd.)
Kraus Process Inc., 110 W. 40th St., New York, N. Y. (J. Tyrell Cheney, Pres.)
Krauss, John S. (See Winslow Construction Co.)
Krause, G. E., Juneau, Alaska
Krause, L. B., 239 S. La Salle St., Chicago, Ill.
Krehbiel, B. F. (See Cement Stone & Supply Co.)
Kreidler, Carl L., Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.
Kremers, Ernest (See Wright & Kremers, Inc.)
Kressly, Paul E., 1780 Ramiro Road, San Marino, Calif.
Krewatch, Wm., 13 S. Ann St., Lancaster, Pa.
Kriege, Herbert F., 1219 W. Bancroft St., Toledo, Ohio
Krier, George H., 814 E. 94th St., Brooklyn, N. Y.
Kronkvist, Hans, Aktiebolaget Skanska Cementgjuteriet, Malmo, Sweden
Kuan, F. C., 118 Cook St. Ithaca, N. Y.
Kubitz, Fred, 427 Oliver Bldg., Pittsburgh, Pa.
Kuemmerle, Clarence E. (See Artcrete Products Co.)
Kuhlman Bldrs. Supply & Brick Co., The, 1428 Nicholas Bldg., Toledo, Ohio (C. E. Kuhlman)
Kuhlman, C. E. (See The Kuhlman Bldrs. Supply & Brick Co.)
Kynnersley, T. R. S., c/o Concrete Association of India, E-2 Clive St., Calcutta, India
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Laboratorio Prove Materiali, Della R. Scuola di Ingegneria, Piazza Cavour 4, Milano 113, Italy (Giulio Revere)
La Camera, Peter, 48 Chandler Ave., Detroit, Mich.
***Laclede Steel Co.,** 1317 Arcade Bldg., St. Louis, Mo. (W. W. Scott)
LaDue, R. M. (See McCracken Machinery Co.)
Lagaard, M. B., c/o Minneapolis Bridge Co., 701 Metropolitan Life Bldg., Minneapolis, Minn.
Lago Petroleum Corp., Apartado 172, Maracaibo, Venezuela (R. L. Mitchell, Ch. Engr.)
Lain, John Sayers-, 133 15th Ave., E., Calgary, Alta., Canada
Lake Ports Supply Co., The, 1917 Penobscot Bldg., Detroit, Mich.
Lake, Simon, East Main St., Stratford, Conn.
Laker Concrete Works, Dalton, Mo. (H. J. Laker)
Laker, H. J. (See Laker Concrete Works)
Lakewood Engineering Co., 520 Dublin Ave., Columbus, Ohio (Lion Gardiner, Vice-Pres.)
LaLonde, Jean Paul, 2042 Marlowe, Ave. Montreal, Canada
Lamar Pipe & Tile Co., 431 Michigan Trust Bldg., Grand Rapids, Mich. (C. E. Edwards, Gen. Mgr.)
Lamb Co., Robert E., 843 North 19th St., Philadelphia, Pa. (Robert E. Lamb)
Lamb, Robert E. (See Robert E. Lamb Co.)
Lambert, Edward A. (See Fletcher, Thompson, Inc.
Lambie, J. Edward, 160 East 156th St. & Lake Shore Dr., Cleveland, Ohio
Lambie, Joseph S., 703 Diamond Bank Bldg., Pittsburgh, Pa.
Lancaster Concrete Tile Co., Lancaster Pa. (Henry Boettcher, Pres.)
Landa, German, Ave. P. Carranza No. 56, Coyoacan, D. F., Mexico
Lander, R. S., Box 196, Little Rock, Ark.
Landor, Edward J., 634 Renekert Bldg., Canton, Ohio
Lane, H. A., Baltimore & Ohio Central Bldg., Baltimore, Md.
Lang, F. C. (See Minn. Highway Dept.)
Lang, Jr., Philip George, 1300 Baltimore & Ohio Bldg., Baltimore, Md.
Langdon, E. W., c/o Joseph T. Ryerson & Son Inc., 2558 West 16th St., Chicago, Ill.
Langeler, M. Bert, 3041 E. 79th Place, Chicago, Ill.
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Large, G. E., 776 S. Remington Road, Columbus, Ohio
Larkin, Chas. W., Carnegie Institute of Technology, Pittsburgh, Pa.
Larsen-Morgan Co., 6th St. at Iowa, Oshkosh, Wis. (R. W. Morgan)
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Larson, Guy H., Materials Department, Wisconsin Highway Comm., Madison, Wisc.
Larson, Louis J., 3306 N. Shepard Ave., Milwaukee, Wisc.
"La Tolteca" Cia. de Cemento Portland, S. A., Independencia 8, Post Office Box 233, Mexico, D. F. (G. H. E. Vivian, Gen. Mgr. & Treas.)
Laubscher, R. E. (See The Sam W. Wmerson Co.)
Lavelle, J., General Assurance Bldg., Toronto, Ont., Canada
Lavoie & Delisle, Chicoutimi, Que., Canada
***Lawrence Portland Cement Co.,** 270 Broadway Ave., New York, N. Y. (E. P. Haubert)
Lawson, David, 703 Broadway, New York City
Lawson, Thomas R., Dept. of Civil Engineering, Rennsseler Polytechnic Inst., Troy, N. Y.
Lea, William S., 1226 University St., Montreal, Canada
Ledyard, H. A. (See Ashland Vault Inc.)
Leonard, W. H., c/o Riverside Cement Co., 621 S. Hope St., Los Angeles, Calif.
Le Soliditit Belge, 10, Rue du Montiteur, Brussels, Belgium (I. Molle)
Le Soliditit Francais, 55 Rue de Lyon, Paris, France
Lee, Kung, c/o Building Sect., Anderson, Meyer & Co., Ltd., Yuen Ming, Yuen Road, Shanghai, China
Lee, Smith & VanDervoort, 15 N. 11th St., Richmond, Va. (Jameson VanDervoort)
Lee, W. S., Power Bldg., Charlotte, N. C.
Leffler, Ralph R., 7021 Oriole Ave., Chicago, Ill.
*Lehaney, J. A. (See The Kansas Portland Cement Co.)
Lehigh Portland Cement Co., Young Bldg., Allentown, Pa.
***Lehigh Portland Cement Co.,** Allentown, Pa. (E. M. Young, Pres.)
***Lehigh Portland Cement Co.,** Allentown, Pa. (E. M. Young, Pres.)
***Lehigh Portland Cement Co.,** Young Bldg., Allentown, Pa.
Lehner, Walter J., Ullrich Bank Bldg., Mt. Clemens, Mich.
Lenhardt, L. G., 2688 Lawrence Ave., Detroit, Mich.
Leon, Carl L. (See Vaso Mfg. Co.)
Leonard, John B., Bureau of Building Inspection, Room 273, City Hall, San Francisco, Calif.
Lepisto, A. (See Waasan Rautabetoni O.Y.)

Lesley, R. W., 611 Penna. Bldg., Philadelphia, Pa.
Leslie Engineering Co., 820½ Felix St., St. Joseph, Mo. (J. R. Leslie)
Leslie, J. R. (See Leslie Engineering Co.)
Levison, Arthur A., c/o Blaw-Knox Co., P. O. Box 915, Pittsburgh, Pa.
Lewis Asphalt Engineering Corp., 30 Church St., New York, N. Y. (R. W. Lewis)
Lewis, H. J., City Hall, Muskegon, Mich.
Lewis, R. W. (See Lewis Asphalt Engineering Corp.)
Ley & Co., Inc., Fred T., Springfield, Mass. (T. A. Pearson)
Li, Dr. Shu-t'ien, Chiao Tung University, College of Civil Engineering, Tangshan, North China
*Lichtenberg, E. H. (See Koehring Co.)
Licuanan, Jose G., P. O. Box 1589, Manila, Philippine Islands
Lieberman, E. (See Lieberman & Hein)
Lieberman & Hein, 190 N. State St., Chicago, Ill. (E. Lieberman)
Liebeskind, Morris Moe, 1931 Walton Ave., Bronx, New York City
Liggett, J. J. (See Van Camp Stone Co.)
Lilly, J. A., Rural Route No. 7, Kalamazoo, Mich.
Lilly, Scott B., Swarthmore College, Swarthmore, Pa.
Lincoln, R. W., 602 Burke Bldg., Seattle, Wash.
Lindau, A. E., Amer. System of Reinforcing, Union Trust Bldg., Chicago, Ill.
Lindon, James J., 932 78th St., Brooklyn, N. Y.
Lindsay, G. L., Universal Atlas Cement Co., 208 S. LaSalle St., Chicago, Ill.
Lindsay & Co., Inc., W. W., 902 Harrison Bldg., Philadelphia, Pa. (James C. Newlin)
Lindsley Co., C. E., 888 Clinton Ave., Irvington, N. J. (C. E. Lindsley, Treas & Mgr.)
Lindsley, C. E. (See C. E. Lindsley Co.)
Linenthal, Mark, 333 Washington St., Boston, Mass.
Linnell, Frank A., Hartford Concrete Co., 97 Vine St., Hartford, Conn.
Liskow, Fred L., 601 West 10 Mile Rd., Pleasant Ridge, Detroit, Mich.
Litter, F. J., The Frederick Snare Corp., 114 Liberty St., New York, N. Y.
Little, C. E. (See Super Concrete Emulsions, Ltd.)
Livermore, A. C., Wilmerding, Pa.
Lock Joint Pipe Co., Ampere, N. J. (A. M. Hirsch, Pres.)
Lock Joint Pipe Co., Ampere, N. J. (J. E. Longley)
Lock Joint Pipe Co., Ampere, N. J. (F. F. Longley)
Locke, Clyde E., 207 Colvin Ave., Buffalo, N. Y.
Lockwood Greene Engineers, Inc., 24 Federal St., Boston, Mass. (H. E. Cousins)
Loeb, II, Henry, Loeb Stone Co., Memphis, Tenn.
Loeffler, H. S., Great Northern Ry. Co., St. Paul, Minn.
Logan, Edgar H., 3504 Troost Ave., Kansas City, Mo.
Logeman, R. T., 208 South La LaSalle St., American Bridge Co., Chicago, Ill.
Lohmann, H. W. (See James Stewart & Co., Inc.)
Loizeaux Lumber Co., J. D., 861 South Ave., Plainfield, N. J. (George B. Coale)
Lomer, G. E. (See McGill University Library)
***Lone Star Cement Co.,** 225 South 15th St., (H. E. Hilts)
***Lone Star Cement Co.,** Sante Fe Bldg., Dallas, Texas (L. R. Ferguson)
***Lone Star Cement Co.,** Birmingham, Ala. (J. W. Johnston)
***Lone Star Cement Co., Virginia, Inc.,** Norfolk, Va. (Dwight Morgan)
***Lone Star Cement Co.,** 1120 Hibernia Bank Bldg., New Orleans, La. (Scott Thompson)
***Lone Star Cement Co., New York, Inc.,** 100 State St., Albany, New York (D. P. Cooper)
Longfellow, D. W. (See Elk River Concrete Products Co.)
Long, James B., 57-59-61 Boyer Arcade, Norristown, Pa.
Longley, J. E. (See Lock Joint Pipe Co.)
Long, J. F., 1365 Cass Ave., Detroit, Mich.
Longley, F. F. (See Lock Joint Pipe Co.)
Lord, Arthur R., 140 S. Dearborn St., Chicago, Ill.
Loring, H. D., (See Ferro Concrete Construction Co.)
Loro & Parisini, 46 Via Manzioni, Milano, Italy
Los Angeles Harbor Department, Berth 161, Wilmington, Calif. (G. F. Nicholson)
Los Angeles, Dept. of Water and Power, P. O. Box 497 M. O., Los Angeles, Calif. (E. F. Scattergood)
Los Angeles City Engineering Dept., 826 Yale St., Los Angeles, Calif. (T. A. Fitch, Engr.)
Los Angeles Public Library, 530 So. Hope St., Continuations Division, Los Angeles, Calif. (Albert C. Read)
Los Angeles Public Library, Continuations Division, 530 S. Hope St., Los Angeles, Calif. (Albert C. Read)
Los Angeles Testing Laboratory, 1300-08 So. Los Angeles St., Los Angeles, Calif. (H. H. McCall)
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Loughland, G. E. (See Northern States Power Co.)
Loughlin, L. J. (See Toledo Concrete Pipe Co.)
***Louisville Cement Co.,** 315 Guthrie St., Louisville, Ky. (W. S. Speed, Pres)
Love, Harry J., 937 Leader Bldg., Cleveland, Ohio
Lovejoy, C. H. (See The Forest City Testing Laboratory)
Lovering-Longbotham Co., 605 Builders Exchange, St. Paul, Minn. (Harry D. Lovering)
Loving, M. W. (See American Concrete Pipe Assn.)
Lowell, J. W., 7402 S. Ashland Ave., Chicago, Ill.
Lubin, Frank, 3432 W. Monroe St., Chicago, Ill.
*Lucas, E. P. (See Superior Portland Cement Co.)
Lucas, J. W., Dept. of Public Works Testing Laboratories, West Block, Ottawa, Canada
Lucchetti, A., Guayama, Porto Rico
Lundeen, Curt (See Axel-Carslon Co.)
Lundoff-Bicknell Co., The, 2024 B. F. Keith Bldg., Cleveland, Ohio (C. W. Lundoff)
Lundoff, C. W. (See Lundoff-Bicknell Co.)
Lundteigen, A., V. P., Ash Grove Lime & Portland Cement Co., Kansas City, Mo.
Luzerne County Road and Bridge Dept., Wilkes-Barre, Pa. (Robert L. Williams)
Lynam, C. G., Turkish Petroleum Co. Ltd., Fields Headquarters, Tuz-Khurmatli, Mesopotamia
Lynch, Charles A. (See The Foote Co. Inc.)
Lynch Co., W. J., 844 Rush St., Chicago, Ill

Lynds, C. D. (See The Sandstone Quarries Co.)
Lyons, R. P., Ready Mixed Concrete Co., 25th & Summit Sts., Kansas City, Mo.
Lyse, Inge, c/o Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.
McBeth, R. W., Kingston, Ohio
McBurney, J. W., 414 Taylor St., Chevy Chase, Md.
McCall, H. C. (See Concrete Engineering Service Co.)
McCall, H. H. (See Los Angeles Testing Laboratory)
McCanless, W. F. (See Ornamental Stone Co.)
McCann, Wm. B. (See Australian Cement Ltd.)
McCarty, M. I. (See Electric Tamper & Equipment Co.)
McCarthy, T. V., Dominion Construction Co. Ltd., Abitibi Canyon via. Cochrane, Ont., Canada
McCaully, W. H., 618 W. Chicago Ave., Chicago, Ill.
McClure, D. A. (See Jamestown Block & Tile Co.)
McColl, J. R. (See Thomas E. Murray Inc.)
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McCoy, R. R. (See George B. Smith Chemical Works, Inc.)
McCracken Machinery Co., Ninth & Division Sts., Sioux City, Ia. (R. M. La Due)
McCracken, Jas. F. (See American Builders Supply Co.)
McCrary Engineering Corp., J. B., Atlanta, Ga. (E. C. Catts, Purch. Agent)
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*McDougall, P. S. (See Ottawa Silica Co.)
McDowell, C. T. J. (See Empire Stone Co. Ltd.)
McEverlast Inc., 111 W. Seventh St., Los Angeles, Calif. (A. M. Shenk, Pres.)
McFarland, William G., 45 East 17th St., Room 1701, New York, N. Y.
McGill University Library, 65 McTavish St., Montreal, Que., Canada (G. E. Lomer, Librarian)
McGillivray, R. (See Duntile Co., Ltd.)
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McKay, W. S., 3317 No. 28, Tacoma, Wash.
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McKee, S. C. The Highway Materials Co., 1604 Toledo Trust Bldg., Toledo, Ohio
*McKelvy, F. G. (See Alpha Portland Cement Co.)
McKenzie, A. H., Griffith-McKenzie Bldg., Fresno, Calif.
McKenzie, C. L. (See Duquesne Slag Products Co.)
McKenzie, B. Stuart (See Canadian Engineering Standards Assn.)
McKenzie, J. G., 1630 Marshall St., Houston, Texas
McKenzie, N. H. (See Bedford Construction Co.)
McKesson, Claude L., 200 Bush St., San Francisco, Calif.
McKinney, J. Harold, St. John Harbour Commission, St. John, N. B., Canada
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McKown, G. W., American System of Reinforcing, 953 Empire Bldg., Milwaukee, Wisc.
McLean, William King, 8 Spring St., Sydney, New South Wales, Australia
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McMillan, E. C., 107 Clifford St., Detroit, Mich.
McMillan, Franklin R., 33 W. Grand Ave., Chicago, Ill..
McMorrow, John T. (See Boston Transit Mixers, Inc.)
McNiven, J. A., 2 Canal St., New Orleans, La.
McNulty, Joseph A., 114 Liberty St., New York, N. Y.
McQuestion, Hubert (See N. Shore Cement Burial Vault Co.)
McRae Steel Co., 509 Free Press Bldg., Detroit, Mich. (K. C. McRae)
McRae, K. C. (See McRae Steel Co.)
McThenia, A. W., Acme Limestone Co., Alderson, W. Va.
McWilliam, R. J., Exton House, Queen St., Brisbane, Australia
McWilliams, R. (See Republic Fireproofing Co., Inc.)
Mabie, H. W., Jr., (See Immel Construction Co.)
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MacGlashan, W. F. (See The MacGlashan Associates)
Macgowan, Ernest S., Box 2075, Minneapolis, Minn.
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MacMillan, A. B., Aberthaw Company, 80 Federal St., Boston, Mass.
Mack, Thomas, 6610 S. Menard Ave., Chicago, Ill.
Mack, W. W. (See Delaware State Highway Dept.)
Mackie, Jas. A., J. & W. Henderson Ltd., 82 Market St., Aberdeen, Scotland
Mackie, Thos. W., c/o Morrow & Beatty Ltd., Fitzroy Harbor, Ont., Canada
Mackenzie, W. F. (See Pretoria Portland Cement Co., Ltd.)
MacKenzie, W. S., Smith, Hinchman & Grylls, Marquette Bldg., Detroit, Mich.
Mackney, A. E., Mgr., Macdonald & Macdonald, 923 Birks Bldg., Vancouver, B. C., Canada
MacNaughton, M. F. (See Milton Hersey Co., Ltd.)
MacNaughton Co., Inc., P. J., 755 Boylston St., Boston, Mass. (P. J. MacNaughton)
MacNaughton, P. J. (See P. J. MacNaughton Co. Inc.)

Macneal, LeRoy C., 284 S. Columbus Ave., Mt. Vernon, N. Y.
Macomber, C. Clark, 38 Chauncy St., Boston, Mass.
Made-Rite Products Co., 5223 McKissock Ave., St. Louis, Mo. (Robert Fullerton, V. P.)
Madison Silo Co., 719 Beaver Bldg., Madison, Wis. (C. C. Woody)
Maeser, Mieth, Room 1-212, Massachusetts Institute of Technology, Cambridge, Mass.
Magny, A. V., Plateau St. Hillaire, Grasse A. M., France
Maguire Co., Chas. B., 507 Grosvenor Bldg., Providence, R. I. (Chas. A. Maguire)
Maguire, Chas. A. (See Chas. B. Maguire Co.)
*Maguire, E. J. (See Manitowoc Portland Cement Co.)
Main, Charles T., 201 Devonshire St., Boston, Mass. (Chas. T. Main, Prop.)
Main Roads Board of N. S. W., 309-315 Castlereagh St., Sydney, New South Wales, Australia (T. H. Upton)
Maine Concrete Co., Main St., Brunswick, Me. (W. A. Jack)
Maine, University of, Technology Experiment Station, Orono, Maine.
Mair, I. J., c/o Chief Civil Engineer, South African Railways, Johannesburg, S. Africa
Malmrot, Hakan, 32 Sturegaton, Grebro, Sweden
Malone, John A., Lancaster, Pa.
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Mann, H. C. (See Union Pacific Systen)
Manning, J. J., Public Works Dept., Navy Yard, Pearl Harbor, Hawaii.
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Mantelli. I. (See Ing. Mantelli, Reg. Corbella & Co.)
Many, Ben J., 1604 Syndicate Trust Bldg., St. Louis, Mo.
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Marcus, M. S. (See Chicago Art Marble Co.)
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Mariner, W. Seward, 28 Pelham St., Newton Center, Mass.
Markland, M. B., Schwehm Bldg., Atlantic City, N. J.
Marquette Cement Mfg. Co., LaSalle, Ill. (C. M. Butler, Chief Chemist)
***Marquette Cement Mfg. Co.,** Marquette Bldg., Chicago, Ill. (R. R. Coghlan, Chemical Engr.)
Marsden, R. W., 16 Norwood Rd., Headingley, Leeds, York, England
Marsh, Earl G., 922 Maryland Ave., N. E., Washington, D. C.
Marsh, Henry F. (See Consolidated Materials Corp.)
Marsh, V. L. (See Wurster Construction Co.)
Marshall, Jr., Jos. M., 1101 Hurt Bldg., Atlanta, Ga.
Marshall, Thos. W., 1341 Connecticut Ave., Washington, D. C.
Marshall, W. V., 1230 Olivia Ave., Ann Arbor, Mich.
Marston, Frank A. (See Metcalf & Eddy)
Martens, Louis H. (See Humboldt Mfg. Co.)
Martel, R. R. (See Calif. Inst. of Technology)
Martin, Edgar, 180 N. Michigan Ave., Chicago, Ill.
Martin, Evan S., 16 Saulter St., Toronto, Ont.
Martin, F. J., 315 Avenue Bldg., Saskatoon, Sask., Canada
Martin, Juan San, Box 630, Havana, Cuba
Martinez, Jose Ricardo, Prado 111, Havana, Cuba
Maryland, University of, Engineering College, College Park, Md. (A. N. Johnson, Dean)
Mason, Frank H., New England Public Service Co., Augusta, Maine
Mason, Philip A. (See City of Hartford)
Mason, Roscoe J., 3221 Lenox Ave., Detroit, Mich.
Masonite Corp., 111 W. Washington St., Chicago, Ill. (E. B. Bushnell)
***Massey Concrete Products Co.,** 968 Peoples Gas Bldg., Chicago, Ill. (E. C. Alexander)
Master Builders Co., The, 7016 Euclid Ave., Cleveland, Ohio (S. W. Flesheim)
Masters, F. M., Const. Engr., 501 Keystone Bldg., Harrisburg, Pa.
Master, R. S., Solakoti Bldg., Grant Road Corner, Bombay, India
Materials, Inc., 600 Garland Bldg., 740 S. Broadway, Los Angeles, Calif. (E. C. Pohlman)
Math, Earle R., 1400-80 John St., New York, N. Y.
Matlin, W. S., State Electrotechnical Trust, c/o J. G. E. Co., 1 River Road, Schenectady, N. Y.
Matthews, Howard B., 73 Newfield St., East Orange, N. J.
Matthews, L. H., Stone & Webster Eng. Corp., 49 Federal St., Boston, Mass.
Mattison, Jr., Geo. A. (See Woodstock Slag Corp.)
Mattison, Richard V. (See Asbestos Shingle Slate & Sheathing Co.)
Mauro, Francesco, 4437 Chestnut St., Philadelphia, Pa.
Maul Co., The, 1640 E. Hancock Ave., Detroit, Mich. (Bruce F. Carty, Secy.)
Mavis, F. Theodore, Engineering Hall, Iowa City, Ia.
Maynard, Arthur J., Mass. St. Farm, State Farm, Mass.
Mayr, J. T., 191-40 114 Ave., St. Albans, N. Y.
Meacham, J. C. (Universal Atlas Cement Co.)
Mead, C. A., 165 Wildwood Ave., Upper Montclair, N. J.
Mead-Suydam Co., 400 Sheridan St., Orange N. J. (F. J. Mead, Pres.)
Mead, F. J. (See Mead-Suydam Co.)
Meade, P. F., Dist. Engr. Portland Cement Assn., Denver, Colo.
Mebus, Charles F., (See Albright & Mebus)
Meckley, E. W., City Hall, Allentown, Pa.
***Medusa Portland Cement Co.,** 1002 Engineers' Bldg., Cleveland, Ohio (R. E. Withrow, Mgr.)
Meier, H. W. (See Donaldson & Meier)
Meldrum, T. J. (See American Memorial Co.)
Meltzer, Joseph, 95 Fenimore Road, New Rochelle, N. Y.
Melvin, Russell C., New Castle Steel Works Fces, New Castle, Pa.
Mepham & Co., Geo. S., 20th & Lynch St., E. St. Louis, Ill. (L. K. Ayers, V. P.)
Menefee, F. N., 104 West Eng. Bldg., Ann Arbor, Mich.
Mercer, L. Boyd, c/o Henry Simon Ltd., Cheadle Heath, Stockport, Cheshire, England
Merchant, Archie W., 728 Hospital Trust Bldg., Providence, R. I.
Merlo, Louis Alvin (See Merlo, Merlo & Ray Ltd.)

Merlo, Merlo & Ray, Ltd., Ford, Ont., Canada (Louis Alvin Merlo, Pres.)
Merrill, B. S., 5133 7th Court So., Birmingham, Ala.
Merriman, Thaddeus, Room 1133, 346 Broadway, New York, N. Y.
Messinger Supply Co., Easton, Pa. (Dr. Wm. Mock, Secy.-Treas.)
Metcalf & Eddy, 1300 Statler Bldg., Boston, Mass. (Frank A. Marston)
Metropolitan Concrete Co., Western Reserve Bldg., Superior & W. 9th St., Cleveland, Ohio (W. H. Crangle)
Metzger-Richardson Co., Century Bldg., Pittsburgh, Pa. (F. L. Metzger)
Metzger, F. L. (See Metzger-Richardson Co.)
Meyer, A. A., Penna. Water & Power Co., Lexington Bldg., Baltimore, Md.
Meyer, B. F., 16 Raycroft Ave., Pittsburgh, Pa.
Meyer, B. A. (See Meyer, Morrison & Co.),
Meyer, C. Louis, 1141 No. 11th St., Omaha, Nebr.
Meyer, Geo. W. (See The Ross-Meyer-Hecht Co.)
Meyer & Holler Inc., 315 Wright & Callender Bldg., Los Angeles, Calif. (R. Howard Armin)
Meyer, Morrison & Co., 39 Cortlandt St., New York, N. Y. (B. A. Meyer, Pres.)
Meyer, M. W. (See Anti-Hydro Waterproofing Co.)
Michigan Concrete Co., 14445 Hamilton Ave., Detroit, Mich. (E. H. Bingham)
Michigan State College, Civil Engineering Dept., East Lansing, Mich. (C. L. Allen)
Michigan, University of, Transportation Library, East Engineering Bldg., Ann Arbor, Mich. (R. L. Morrison)
Miesenhelder, P. D., c/o Indiana State Highway Comm., Woodruff Pl., Indianapolis Ind.
Milburn Lime & Cement Co., Ltd., 59 Crawford St., Dunedin, N. Z. (J. H. Stewart, Gen. Mgr.)
Milgram, Joseph, (See Nelson Pedley Construction Co.)
Millen, Vincent, c/o Chirigui Land Co., Puerto Armuelles, Rep. of Panama, C. A.
Miller, D. Traver, (See Industrial Engineering Co.,)
Miller, F. E. (See Standard Slag Co.)
Miller, G. R., S. M. Damon Bldg., Honolulu, Hawaii
Miller, J. G. (See Great Lakes Concrete Pipe Co., Inc.)
Miller, L. C., 610 Merchants Bank Bldg., Indianapolis, Ind.
Miller, N., c/o Beauharnois Construction Co., Box 50, Beauharnois, Que., Canada
Miller, O. H. (See Hollywood Concrete Pipe Co.)
Millhouse, C. C. (See The Automatic Sealing Vault Co.)
Milligan, Mr. (See John V. Gray Construction Co.)
Mills, Jr., W. H. (See S. C. State Highway Dept.)
Milwaukee Concrete Products Cooperative Assoc., 87 27th St., Milwaukee, Wisc.
Miner, Joshua L., c/o The Atlas Lumnite Cement Co., Chrysler Bldg., 135 East 42nd St., New York, N. Y.
Miner, Reginald B., 89 Jewett St., Newton, Mass.
Minneapolis Street Ry. Co., 1 So. 11th St. Minneapolis, Minn. (O. F. Moore)
Minnesota Highway Dept., 1246 University Ave., St. Paul, Minn. (F. C. Lang, Engr. of Tests and Inspection)
Minnesota Mining & Mfg. Co., Forest & Fauquier Sts., St. Paul, Minn. (L. A. Hatch)
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Minton-Scobell Co., The, 679 The Arcade, Cleveland, Ohio (J. H. Minton)
Minton, J. H. (See The Minton-Scobell Co.)
Minwax Company, Inc., 11 West 42nd St., Allrich S. Harrison, Vice-Pres.)
*__Missouri Portland Cement Co.,__ 13th Fl. S. W. Bell Telephone Bldg., St. Louis, Mo. (H. C. Block, Pres.)
Missouri State High Dept., Jefferson City, Missouri (F. V. Reagel, Engr.)
Mitchell, James, 26 Journal Square, Jersey City, N. J.
Mitchell, Nolan D., 134 Beech St., S., Clarendon, Va.
Mitchell, R. L. (See Lago Petroleum Corp.)
Mitchell, Thos. H , c/o The H. K. Ferguson Co., Marunouchi Bldg., Tokyo, Japan
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Mizoguchi, K. (See Onoda Cement Co. Ltd.,)
Mock, Dr. Wm. (See Messinger Supply Co.)
*Mogensen, O. E. (See F. L. Smidth & Co.,)
Mohr, H. A., 1865 Beacon St., Waban, Mass.
Molfino, Juan P., Lavalleja 1820, Montevideo, Uruguay
Molle, I (See Le Soliditit Belge)
Monarch Engrg. Co., Chamber of Comm. Bldg., Buffalo, N. Y. (H. R. Wait, Pres.)
Moncrieff, Alan (See Concrete Association of India)
Monks & Johnson, 99 Chauncy St., Boston, Mass. (B. A. Rich)
Montz, A. S., 740 Poydras St., New Orleans, La.
Mooney, Thomas G. (See North American Cement Corp.)
Moorby. J. Howard, Scott & Urner, Caixa Postal, 1863, Rio de Janeiro, Brazil
*Moore, E. J. (See Turner Const. Co.)
Moore, Jay (See American Reduction Co.)
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Moore, O. F. (See Minneapolis Street Ry. Co.)
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Moore, Thos. V., c/o Raymond Concrete Pile Co., 1435 Gorsuch Ave., Baltimore, Md.
Moran Construction Co., The, 2123 So. 56th St., Omaha, Neb. (Frank Moran, Pres.)
Moran, Frank, (See The Moran Construction Co.)
Moran, Robert L., 9 Wellington St., W., Toronto 2, Ont., Canada
Moran, W. C. (See Riehle Bros. Testing Machine Co.)
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Moreell, Ben, Puget Sound Navy Yard, Bremerton, Wash.
Moran & Proctor, 342 Madison Ave., New York, N. Y. (G. L. Freeman)
*Morgan, Dwight (See Lone Star Cement Co., Virginia, Inc.)
Morgan, J. Grant, 403 Inter-Island Bldg., Honolulu, Hawaii
Morgan, N. D., 306 Architectural Bldg., Urbana, Ill.
Morgan, Nathan W., 4115 Wisconsin Ave., Washington, D. C.
Morgan, R. W. (See Larsen-Morgan Co.)
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Morrill, George P., Westbrook, Conn.

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Morris, Clyde T., Ohio State University, Columbus, Ohio
Morris, Mark, c/o Iowa State Highway Commission, Ames, Iowa
Morris, S. B. (See Pasadena Water Dept.)
Morrison, Phillip (See Pioneer Sand & Gravel Co.)
Morrison, R. L. (See Univ. of Michigan)
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Morse, Floyd C., 2480 Tuxedo, Detroit, Mich.
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Moto, Candelario Calor P. O. Box 131, Marina Station, Mayaguez, P. R.
Mott, John (See Clinton Metallic Paint Co.)
Moulton, A. G., 1028 Monroe Bldg., Chicago, Ill.
Mouritz, C., Victoria House, 117 Victoria St., London, S. W. I., England
Moyer Albert, (See Vulcanite Portland Cement Co.(
Mozzicato, James S., 47 Hancock Ave., Medford, Mass.
Moyer, Wm. N., 921 Washington Ave., Northhampton, Pa.
Mueller, Geo. W. (See The Northwestern Terra Cotta Co.)
Mueller, Harold P., 204 Oak Lane Trust Bldg., Broad & 67th St., Philadelphia, Pa.
Mueller, J. W., Palladium Bldg., Richmond, Ind.
Mueser, William H., 342 Madison Ave., Room 1126 A., New York, N. Y.
Mueser, Wm. (See Concrete Steel Engrg. Co.)
Muhlenberg, Charles R., 1020 Center Ave., Reading, Pa.
Muirhead Construction Co., Wm., Durham, N. Car. (Wm. Muirhead)
Muirhead, Wm. (See Wm. Muirhead Construction Co.)
Mulroy-Cooke Co., Inc., 636 Ogden St., Newark, N. J. (E. J. Mulroy, Pres.)
Mulroy, E. J. (See Mulroy-Cooke Co., Inc.)
Mumm, Jr., Hans (See Everett Concrete Products Co.)
Munsell, A. W., 999 West Side Ave., Jersey City, N. J.
Munsell, J. R., 269 Cherry Lane, Teaneck, N. J.
Muntz, E. P., 403 Lehigh Valley Terminal, Buffalo, N. Y.
Murdough, J. H. (See Texas Technological College)
Murphy, F. E. (See Stevens & Wood, Inc.)
Murphy, Frank J. (See Valentine Lumber & Supply Co.)
Murphy, Glen, 201 Engineering Hall, University of Illinois, Urbana, Ill.
Murray, Andrew, County Engineer, Waitemata County Council, Auckland, N. Z.
Murray, Inc. Thomas E., 88 Lexington Ave., New York, N. Y. (J. R. McColl)
Muth, G. M. (See Berks Building Block Co.)
Muths, George A., P. O. Box 293, Mobile, Ala.
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Mylchreest, George Lewis, 238 Palm St., Hartford, Conn.
Mylrea, T. D., 1707 Koppers Bldg., Pittsbugh, Pa.
Myren, B. J. (See N. P. Severin Co.)
Myron, J. A., c/o Cincinnati Union Terminal Co., 1020 Temple Bar Bldg., Cincinnati, Ohio
National Fireproofing Co., 122 E. 42nd St., New York, N. Y. (Philip Bevier)
National Lime Association, 927 Fifteenth St. N. W., Washington, D. C. (W. V. Brumbaugh)
National Paving Brick Mfgrs. Assn., National Press Bldg., Washington, D. C. (G. F. Schlesinger)
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Nailcrete Corp., The, 105 W. 40th St., New York, N. Y. (F. B. DeGress)
Naito, Tachu, Univ. of Waseda, Tokyo, Japan.
Nash, Jack T., 5416 Victor St., Dallas, Texas
Nashville Breeko Block & Tile Co., 306 Cowan St., Nashville, Tenn. (Chas. Wm. Akers, Mgr.)
***Nassau Sand & Gravel Co.**, 17 Battery Place, New York, N. Y. (W. J. Timberman)
Nasu, Akiya, Mgr., Nakashibuya No. 715, Tokyo, Japan
***National Building Units Corp.**, 1600 Arch Street, Philadelphia, Pa.
National Cement & Block Supply Co., The, Wayne St. at Terminal R. R., Toledo, Ohio (C. B. Fellabaum)
National Crushed Stone Assn., 14th & S Sts. N. W., Washington, D. C. (A. T. Goldbeck)
***National Equipment Corp., The**, 31st & Concordia Aves., Milwaukee, Wis.
National Steel Fabric Co., Union Trust Bldg., Pittsburgh, Pa. (W. H. Shaffer, Jr.)
National Testing Laboratories Ltd., The, 223 James St., Winnipeg, Man., Canada (L. J. Street)
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Neff, Stewart S., 19 Arlington Pl., Buffalo, N. Y.
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Neill, W. A. (See The Dorr Co., Inc.)
Nelson, Ned H. (See Pacific Coast Cement Co.)
Nelsen Concrete Culvert Co., Inc., Box 61, Champaign, Ill. (E. P. Shapland)
Nelson, Dewey, Knollwood Road, Reading, Mass.
Nelson, G. V., 4325 Mercier St., Kansas City, Mo.
Nelson, Norman, Cincinnati, Ohio.
Nelson Pedley Construction Co., 1510 Chestnut St,, Philadelphia, Pa. (Joseph Milgram)
Nepenna Building Materials Co., P. O. Box 427, Kingston, Pa. (F. L. Schott)
Ness, Gilbert, "Stonelea" Harlow Hill, Otley Road, Harrogate, England
Nevin, Thos. M., 9 East 8th St., New York, N. Y.
Newalezada, Esq., A. K. (See Bashie Ahmad Siddique)
Newark Concrete Pipe Co., 323 Broadway, Newark, N. J. (Francis H. Sherrerd, V. P.)
***Newaygo Portland Cement Co.**, Newaygo, Mich.
Newcomb, Rexford, University of Illinois, 118 Architecture Bldg., Urbana, Ill.
Newhard, E. P. (See Pennsylvania Dixie Cement Corp.)
New Jersey Zinc Co., Palmerton, Pa.
Newlin, James C. (See W. W. Lindsay & Co., Inc.)
Newton, A. I. (See Berkshire Gravel Co.)
Nicholson, Geo. E., Pres., National Cement Co., 819 Commerce Bldg., Kansas City, Mo.

Nickel Plate Sand & Gravel Co., 610 Commerce Bldg., Erie, Pa. (H. F. Rath, Sec.)
Nichol, F. E., 756 E. Skidmore, Portland, Oregon
Nichols, Charles E., 49 Federal St., Boston, Mass.
*Nichols, C. E. (See Stone & Webster)
Nichols, G. R., Parkview Hotel, Memphis, Tenn.
Nichols, John R., 45 Newbury St., Boston, Mass.
Nicholson, G. F. (See Los Angeles Harbor Department)
Nielsen, Frederick E., 829 Dobson St., Evanston, Ill.
Nielsen, S. W., Fountain City, Tenn.
Nikitin, P. E., c/o Amtorg Trading Corp., 261 Fifth Ave., New York, N. Y.
Nims, Chas. B., Dist. Engr., Portland Cement Assn., 1009 Failing Bldg., Portland, Ore.
Nishkian, L. H., 525 Market St., San Francisco, Calif.
Nixon, H. O., The Youngstown Sheet & Tube Co., Youngstown, Ohio
Noceto, Luis, Agraciada, 2576, Montevideo, Uruguay, S. A.
Noel-Paton, V. F., c/o Killick Nixon & Co., Home St., Bombay, India.
Noetzli, Fred A., 928 Central Bldg., Los Angeles, Calif.
Noice, Blaine, 5436 Carlton Way, Hollywood, Los Angeles, Calif.
Nolan, S. Frank, City Engr., City Hall, Providence, R. I.
Nonporite Pty. Ltd., 296 Burwood Rd., Hawthorne, Vict., Australia (Norman Griffith)
Noonan, W. H., 334 Roy Bldg., Halifax, Nova Scotia, Canada
Norfolk, Parker F., Onley, Virginia
Noren, Bertil, OR. Dalsland, Sweden
Norred, Jr., C. V., 2115 Jones Ave., Nashville, Tenn.
Norris, Frederick A. (See Emerson-Norris Co.)
Norris, J. F., 107 Norris St., Rochester, N. Y.
Norris, W. H., 5131 Cypress St., Pittsburgh, Pa.
Norsk Portland Cementkonter, Krikegaten 16, Oslo, Norway
***North American Cement Corp.,** 285 Madison Ave., New York, N. Y. (Thomas G. Mooney)
North American Cement Corp., Catskill, N. Y. (G. A. Witte)
***North American Cement Corp.,** Catskill, N. Y. (H. F. Kichline, Chief Chemist)
Northern Construction Co., Ltd., Vancouver Block, Vancouver, B. C., Canada (Wm. Smaill)
Northern States Power Co., 15 South Fifth St., Minneapolis, Minn. (G. E. Loughland)
Northeastern University, 316 Huntington Ave., Boston, Mass. (Henry B. Alvord)
North Jersey Dist. Water Supply Comm., 24 Commerce St., Newark, N. J. (Arthur H. Pratt)
Northwest Concrete Products Assn., 518 Exchange Bldg., Seattle, Wash. (F. R. Zaugg)
North Shore Cement Burial Vault Co., 2503 Buchanan Rd., Kenosha, Wis. (Hubert McQuestion)
Northwest Steel Rolling Mills, Inc., 4315 9th Ave., N. W., Seattle, Wash. (Paul H. Steuding, Sales Mgr.)
Northwestern States Portland Cement Co., Mason City, Iowa (G. C. Blackmore)
Northwestern Terra Cotta Co., The, W. 1st Ave. & Umatilla St., Denver, Colo. (Geo. W. Mueller)
Norton, Paul W., 1330 Statler Bldg., Boston, Mass.
Nova Scotia Power Comm., The, Halifax, Nova Scotia (Harold S. Johnston)
Nowell, Jr., Joseph C., c/o Stone & Webster Engineering Corp., P. O. Box 52, Old Bridge, N. J.
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Nutting Co., of N. J., H. C., 101 High St., Newark, N. J. (Dewey Thompson)
Nutting Co., H. C., 71st St. P. O. Box 1805, Cincinnati, Ohio (C. O. Christy)
Ober, R. H., Thompson Bldg., Seattle, Wash.
Ochs, Erie J. (See Standard Portland Cement Co.)
O'Connor, A. J. (See Koenig Coal & Supply Co.)
Oehmann, John W., 110 District Bldg., Washington, D. C.
Oesterblom, I., 1517 So. Michigan Ave., Chicago, Ill.
O'Fallon, R. S., Apartado 444, Santiago de Cuba, Cuba
Ogilvie, A., (See Hydroprest Products Ltd.)
Ohtomo, Kosuke, c/o Chichibu Cement Co., Chichibu, Saitama-Ken, Japan
Oklahoma A. & M. College, Library, Stillwater, Okla. (Mrs. Elsie D. Hand, Libr.)
Oklahoma Glazed Cement Pipe Co., Tulsa, Okla. (J. M. Chandler, Pres.)
Oklahoma Testing Laboratories, 519½ West Main St., Oklahoma City, Okla. (Wm. Furber Smith)
Okubo, Toshiyuki, 12 Furuichi Machi, Seoul, Korea, Japan
Oliver, Herbert D., 1521 Healey Bldg., Atlanta, Ga.
Oliver, William A., 602 W. Church St., Champaign, Ill.
Olsen, E. (See Badger Concrete Co.)
Olsen, Eugene F. (See Consolidated Concrete Machinery Co.)
Olsen, Ole K., 822 Perdido St., New Orleans, La.
Olympic Portland Cement Co., Ltd., The, P. O. Box 17, Bellingham, Wash. (A. F. Krabbe, Gen. Supt.)
***Olympic Portland Cement Co.,** 4125 Dexter Horton Bldg., Seattle, Wash. (W. P. Cameron)
Olympian Stone Company, 6401 Seaview Ave., Seattle, Wash. (Leo Swartz)
Onoda Cement Co., Ltd., Dairen Branch Works, Dairen, South-Manchuria, China (K. Mizoguchi)
Onoda Cement Co., Yamaguchi-Ken, Japan
Orbeck, Martin J., c/o Holland, Ackerman & Holland, 106 E. Liberty St., Ann Arbor, Mich.
Ornamental Stone Co., Charlotte, N. C. (W. F. McCanless, Pres.)
Ornelas, Samuel, Comision Nacional de Irrigacion, Est La Cruz, Chih., Mexico.
O'Rourke, Chas. E., Lincoln Hall, Ithaca, N. Y.
Orozco, J. Vicente, Estacion La Cruz, Chihuahua, Mexico
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Osakeyhtic Constructor Aktiebolag, Helsingfors, Finland
Osborn Engineering Co., The, 7016 Euclid Ave., Cleveland, Ohio (K. H. Osborn)
Osborn Co., C. J., 132 Nassau St., New York, N. Y. (W. F. Purdy)
Osborn, K. H. (See The Osborn Engineering Co.)
Osborne, Raymond G., Basement, Marsh-Strong Bldg., 9th & Main St., Los Angeles, Calif.
Oscar, L. C., Lehigh Portland Cement Co., Young Bldg., Allentown, Pa.

Osler, Richard B., The Cement Marketing Co., Portland House, Tothill St., Westminster, London, Eng.
*__Ottawa Silica Co.,__ Ottawa, Ill. (P. S. McDougall, Gen. Mgr.)
Ottawa Suburban Roads Commission, 279 Carling Ave., Ottawa, Canada (Alan K. Hay)
Outzen, Andrew N., Supt., River Rouge Co., 415 Clifford, Detroit, Mich.
Overend, A. B., c/o H. Vivian Taylor, 499 Little Collins St., Melbourne, Australia
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Oxley, J. Morrow (See Chapman & Oxley)
Ozaki, K., 599 Otsuka, Kami-Ochiai, Tokyo, Japan
Pacific Coast Cement Co., 3801 East Marginal Way, Seattle, Wash. (Ned H. Nelson)
Pacific Coast Steel Corp., Terminal Box 3147, Seattle, Wash. (W. A. Roth)
Pacific Coast Talc Co., 2149 Bay St., Los Angeles, Calif. (Geo. Ames, Mgr.)
Pacific Stone Co., The, 4257 Eighth Ave., N. W., Seattle, Wash. (Gilbert E. Tucker)
Painter, Geo. M., Belmont Cement Burial Case Co., West Manayunk, Pa.
Palazzo, P., Direccion de Puentes y Caminos, Calle 7 entre 56 y 57, LaPlata, Argentina
Palmer, E. W. (See Riley, Harbord & Law)
Panzer, R. R., 609 Southern Ohio Bank Bldg., Cincinnati, Ohio
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Paquette, J. E., Shawinigan Engineering Co., Ltd., Rapide Blanc, Que., Canada
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Pargas Kalkbergs Aktiebolag, Pargas, Finland
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Parker, Philip M., Julesburg, Colo.
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Patten, Nathan Van (See Stanford University)
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Pearson, T. A. (Fred T. Ley & Co.)
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Pease, F. A., 1211 Terminal Tower Bldg., Cleveland, Ohio
*__Peerless Portland Cement Co.,__ Union City, Mich. (Wm. M. Hatch, Mgr.)
Peerless Stone Ltd., 510 Rhodes Ave., Toronto, Ont., Canada (F. E. Beer)
Peiper, A. S., 2230 N. Lambert St., Philadelphia, Pa.
Pei Yang University, Tientsin, China
Penker Construction Co., The, 1030 Summer St., Cincinnati, Ohio (Irwin A. Penker, Pres.)
Penker, Irwin A. (See The Penker Contr. Co.)
Penniman Concrete & Material Co., 3000 Junius St., Dallas, Texas (Nolan Browne, V. P.)
Pennsylvania-Dixie Cement Corp., 521 Fifth Ave., New York, N. Y. (Blaine S. Smith, Pres.)
Pennsylvania-Dixie Cement Corp., 1010 James Bldg., Chattanooga, Tenn. (E. P. Newhard)
Pennsylvania State Highway Dept., Harrisburg, Pa. (W. H. Connell, Asst. State Highway Comm.)
Pennsylvania Water & Power Co., 1611 Lexington Bldg., Baltimore, Maryland (N. B. Higgins, Asst. Ch. Engr.)
Perfected Burial Vault & Cement Products Co., 2506-10 Irving Pk. Blvd., Chicago, Ill. (H. A. Dahlquist, V. P.)
Perkins, G. Howard, c/o Warren Bros. Co., P. O. Box 1869, Boston, Mass.
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Perrin, Harvey (See Southern Cement Products Co.)
Perrot, Emile G., Boyertown Bldg., 1211 Arch St., Philadelphia, Pa.
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Peters, N. L. (See Peters, Strange & Bradshaw)
Peters, Strange & Bradshaw, 617 Myrick Bldg., Lubbock, Texas (N. L. Peters)
Peterson, Ivan C. (See Suhr, Berryman, Peterson & Suhr)
Peterson, Lawrence E., Osthoff-Peterson Co., 110 E. Wisconsin Ave., Milwaukee, Wis.
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Pettus, L. A., Hydraulic Press Brick Co., 705 Olive St., Room 1000, St. Louis, Mo.
Pharis, L. M., c/o Minnesota Powers & Light Co., Duluth Minn.
Phelan, Frank J. (See Integral Waterproofing Co.)
Phillips, Leroy, 309 W. 9th St., Hutchinson, Kansas
Phillips, R. S., 33 W. Grand Ave., Chicago, Ill.
Phillips, Wm. B., 843 So. Hauser Blvd., Los Angeles, Calif.
Phillips, W. H., 5441 Shafter Ave., Oakland, Calif.
Philpott, James H., 86 Walnut St., Wabash. Ind.
Pierson Co., J. W., 89 Dodd St., East Orange, N. J. (James T. Pierson, Vice-Pres.)
Pierce Testing Laboratories, The, 730-34 19th St., Denver, Col. (George Pierce)
Pierce, George (See Pierce Testing Laboratories)
Pierson, James T. (See J. W. Pierson Co.)
Pierson, George E. (See Indiana Portland Cement Co.)
Pigott, Joseph M., Hamilton, Ont., Canada
Piloff, Albert K., 89 Bainbridge St., Brooklyn, N. Y.
Pioneer Sand & Gravel Co., 901 Harrison St., Seattle, Wash. (Phillip Morrison)

Pirkner, Herbert R., River Edge, N. J.
Pitner, L. E. (See Cincrete Products Corp.)
Pittsburgh, City of, Dept. of City Transit, 906 City County Bldg., Pittsburgh, Pa. (Winters Haydock)
Pittsburgh Testing Laboratory, Stevenson St. & Locust, Pittsburgh, Pa. (A. R. Ellis, Gen. Mgr.)
Place, Harrington (See Detroit Bureau of Government Research)
Plagwit, Eric, Koppers Bldg., Pittsburgh, Pa.
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Pletta, Dan H., Civil Engineering Dept., University of South Dakota, Vermillon, S. Dak.
Plumb, R. A. (See The Truscon Laboratories)
Pohlman, E. C. (See Materials, Inc.)
Polaris Concrete Products Co., Box 86, West Duluth, Minn. (Gordon H. Butler)
Pollak, Julian A. (See The Pollak Steel Co.)
Pollak Steel Co., The, P. O. Box 1461, Cincinnati, Ohio (Julian A. Pollak, Vice-Pres.)
P'Pool & Son, A., 2716 25th St., Meridian, Miss. (A. O. P'Pool)
P'Pool, A. O. (See A. P'Pool & Son)
Porcheddu, G. A. (See Soc. Porcheddu Ing., G. A.)
Porete Mfg. Co., 346 Riverside Ave., Newark, N. J.
Poriss Co., The S. C., 198 N. Quaker Lane, West Hartford, Conn. (S. C. Poriss)
Poriss, S. C. (See S. C. Poriss Co.)
Porter, F. B. (See Southwestern Laboratories)
Porter, John L., Director, Water Purifications Stations, 2142 Eagle St., New Orleans, La.
Porter, J. Miller, c/o Structural Gypsum Corp., Linden, N. J.
Portland Cement Assoc., Hibernia Bank Bldg., New Orleans, La. (W. G. Hudson)
Porto Rico College of Engineering, University of Porto Rico, Mayaguez, Porto Rico (Frank O. Kerr)
Post, C. L. (See Condron & Post)
Potts, A. H., 1216 Architects' Bldg., Los Angeles, Calif.
Pottsville Building Block Co., Pottsville, Pa. (Barton R. Biever)
Powell, F. H., 503 Roosevelt Bldg., 727 W. 7th St., Los Angeles, Calif.
*Powell, F. H. (See Southwestern Portland Cement Co.)
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Powers, Eugene S., 1520 W. Locust St., Philadelphia, Pa.
Powers, J. W. (See Arizona Highway Dept.)
Power, S. M., School of Mines, Bendigo, Victoria, Australia
Powers, Treval C., Portland Cement Assoc., 33 West Grand Ave., Chicago, Ill.
Pozzo, Alberto, Corso Giuseppe Govone N. 6-8, Torino, Italy
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Praeger, Emil, 152 Stratford Rd., Brooklyn, N. Y.
Prairie Concrete Products Co., Drawer P., Prairie Du Chiene, Wis. (G. A. Kahler)
Pratt, Arthur H. (See North Jersey District Water Supply Commission)
Pratt, K. H. (See The Bolton Pratt Construction Co.)
Preis, C. G. (See American Can Co.)
Pretoria Portland Cement Co., Ltd., P. O. Box 3811, Johannesburg, S. Africa (W. F. Mackenzie)
Price, Philip W., 6950 Rosewood St., Homewood Sta., Pittsburgh, Pa.
Price, T. M. (See Kaiser Paving Co.)
Priest, George E. (See Rib-Stone Concrete Corp.)
Priester Construction Co., 1121 American Bank Bldg., Davenport, Iowa (O. F. Priester)
Priester, O. F. (See Priester Construction Co.)
Prince, John, 1805 Grand Ave., Kansas City, Mo.
Probst, Prof. Dr. Ing. E., Vorholzstrasse 2, Schwarzwaldstr., 10, Karlsruhe, i. B., Germany
Proctor, W. D. (See James, Proctor & Redfern, Ltd.)
Props of Hay's Wharf Ltd., Tooley St., London, S. E. I. England (H. J. M. Skidmore)
Proudley, C. E., Munsey Bldg., Washington, D. C.
Providence Transit Concrete Corp., 40 Allens Ave., Providence, R. I.
Punjab Portland Cement, Ltd., Wah, N. W. Railway, Attock District, Punjab, India
Purdy, W. F. (See C. J. Osborn Co.)
Purrington, Wallace F., 19 Thorndike St., Concord, New Hampshire
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Pyramid Company, 433 Maple Ave., Downers Grove, Ill. (W. H. Frazier)
Quaglia, Prof. Giovanni (See Regia Scula D' Ingegneria di Pisa)
Quebec, City of, City Hall, Quebec, Que., Canada (Edouard Hamel, C. E.)
Quebec Dept. of Public Works, Government Buildings, Quebec, Canada (Ivan E. Vallee)
Quebec, Department of Roads, Parliament Buildings, Quebec, Canada (J. L. Boulanger)
Queensland Main Roads Commission, Brisbane, Australia (J. E. England)
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Quinton Code & Hill, Suite 712 Standard Oil Building, Los Angeles, Calif. (Louis C. Hill)
Rackle, E. E. (See The George Rackle & Sons Co.)
Rackle & Sons Co., The Geo., Station D, Cleveland, Ohio (Geo. L. Rackle, V. P.)
Rackle & Sons Co., The George, Box 1778, Station D, Cleveland, Ohio (E. E. Rackle, Pres.)
Rackle, Geo. L. (See The George Rackle & Sons Co.)
Radbill, Ray, Moylan, Pa.
Rader, B. H., Conway Bldg., Chicago, Ill.
Radigan Frank J., Court House, Jersey City, N. J.
Raeder, Warren, University of Colorado, Boulder, Colo.
Rail Steel Bar Association, Builders Bldg., 228 N. LaSalle, Chicago, Ill. (H. P. Bigler)
Ramm Co., E. L., Tilden Ave., LaGrange, Ill. (E. L. Ramm)
Ramm, E. L. (See E. L. Ramm Co.)
Randall, Frank A., 205 West Wacker Drive, Chicago, Ill.
Randolph, Edward Sydney, Designing Engineer, Madden Dam, The Panama Canal, Box 72, Balboa Heights, C. Z.
Rankin, John, Structural Engineer, Deputy City Building Surveyor, Sydney, N. S. W., Australia
***Ransome Concrete Machinery Co.,** Dunellen, N. J. (A. P. Robinson)
Rapp, Paul, Room 151, Industrial Bldg., Bureau of Standards, Washington, D. C.
*Rasmussen, A. C. (See Insley Mfg. Co.)
Ratcliff, T. R., Union Station, Annex, Indianapolis, Ind.
Rath, H. F. (See Nickel Plate Sand & Gravel Co.)

Rawls, E. M., State Highway Dept., Box 92, Hot Springs, N. C.
Rawson, R. A., The Master Builders Co., 7016 Euclid Ave., Cleveland, Ohio
Ray, Kenneth (See Hume Pipe Co. (S. A.) Ltd., The)
Raymond, Chas., University Tower Bldg., St. Catherine St. W. & University St., Montreal, Que., Canada
*__Raymond Concrete Pile Co.,__ 140 Cedar St., New York, N. Y. (H. P. Hamlin)
Raywid, Leo, 1608 K. St., N. W., Washington, D. C.
Read, Albert C. (See Los Angeles Public Library)
Ready Mixed Concrete Co., Quincy Ave., Knoxville, Tenn. (John L. Humberd, Pres.)
Ready Mix and Supply Corp., P. O. Box 404, Albany, N. Y. (N. D. Crowley, Pres.)
Reagel, F. V. (See Missouri Highway Dept.)
Reaney, Charles F., 5341 Goodwin St., Dallas, Texas
Reaves, G. M., c/o St. Andrews' School, Middletown, Del.
Rebolledo, Miguel, 11a Artes 169, Mexico City, Mexico
Reed-Lewis, E. W. (See Super Cement (America) Co., Ltd.)
Reed-Lewis, W. J. D. (See The Super Cement Co.)
Reed Co., William T., 89 Broad St., Boston, Mass. (Wm. T. Reed)
Reed, Wm. T. (See William T. Reed Co.)
Regia Scuola D'Ingegneria, (Biblioteca) Torino, Italy
Regia Scula D'Ingegneria di Pisa, Torino, Italy (Prof. Giovanni Quaglia)
Reichard-Coulston, Inc., 95 Madison Ave., New York, N. Y. (W. A. Kennedy)
Reidy, Maurice A., 44 School St., Boston, Mass.
Reinhold, W. J., Equitable Life Bldg., Queen St., Brisbane, Australia
Reis, F. Santos, Barao Bom Retiro, 58, Rio de Janeiro, Brazil
Remler, R. F. (See The Grasselli Chemical Co., Inc.)
Remley, Norman W., Bureau of Bridges, Mo. State Highway Dept., Jefferson City, Mo.
Rengade, M. (See Ste. Amedes Chaux et Ciments de La Farge et du Teil)
Reniger Construction Co., 127 N. Cedar St., Lansing ,Mich. (M. H. Reniger)
Reniger, M. H. (See Reniger Construction Co.)
Rensaa, E. M., Room 460, Union Station, Winnipeg, Manitoba, Canada
Renwick, E. A. (See Holabird & Root)
Republic Fireproofing Co., Inc., 31 Union Square, New York, N. Y. (R. McWilliams)
Resseguie, H. D., Box 28, Bristol, N. H.
Reuter, Jr., Louis F. (See Paul Rieson's Sons Co.)
Revere, Giulo (See Laboratorio Prove Materiali)
Reynolds, C. E. (See Toronto Ready Mix Concrete Ltd.)
Reynolds, Frank L., 420 Lexington Ave., New York, N. Y.
Reynvaan, A. J., c/o United Engineers & Constructors, 112 North Broad St., Philadelphia, Pa.
Rheinstein Construction Co., Inc., 21 E. 40th St., New York, N. Y. (A. Rheinstein, Pres.)
Rheinstein, A. (See Rheinstein Construction Co., Inc.)
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Rhodes, Geo. I. (See Ford, Bacon & Davis, Inc.)
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Rich, B. A. (See Monks & Johnson)
Rice, E. W. (See Santa Cruz Portland Cement Co.)
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Rickey, James W. (See Aluminum Company of America)
Rickter, Jean Paul (See The Ballinger Co.)
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Riehle Bros. Testing Machine Co., 1424 North 9th St., Philadelphia, Pa. (W. C. Moran)
Rieson's Sons Co., Paul, 2660 N. Humboldt Ave., Milwaukee, Wis. (Louis F. Reuter, Jr.)
Riis, Erling (See Southern Kraft Corp.)
Riley, Harbord & Law, 16 Victoria St., Westminster, London, England (E. W. Palmer)
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Rishel, Jno. (See Ira C. Wilcox, Inc.)
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Ritter, Louis E., 140 So. Dearborn St., Chicago, Ill.
Ritterbush Bros., 209 North 7th St., Bismarck, N. Dak. (Robert A. Ritterbush)
Ritterbush, Robert A. (See Ritterbush Bros.)
*__Riverside Cement Co.,__ 621 S. Hope St., Los Angeles, Calif. (R. C. Aussieker)
Riverton Lime Co., Riverton, Va. (G. W. Hutchinson)
Rivet-Grip Steel Co., The 2404 Prospect Ave., Cleveland, Ohio (H. A. Stuart)
Roach, M. J., Turner Constr. Co., 420 Lexington Ave., New York, N. Y.
Robb, Charles G., Safe Harbor Water Power Corp., Safe Harbor, Pa.
Robb, L. A. (See Ambursen Construction Co., Ltd.)
Robb, R. E., General Motors Inst. of Technology, Chevrolet & 3rd Aves., Flint, Mich.
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Roberts, F. K., Public Works Dept., Karamea, New Zealand
Roberts, Hayden, (See Holliburton Oil Well Cementing Co.)
Roberts, H. W., The Hotel Duncan, 1151 Chapel St., New Haven, Conn.
Roberts & Schaefer Co., 1110 Wrigley Bldg., Chicago, Ill. (E. E. Barrett, Pres.)
Roberts, Warren T. (See Abel Construction Co.)
Robertson, Andrew, Faculty of Engineering, University of Bristol, Bristol, England
Robertson, R. D. (See Independent Concrete Pipe Co.)

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Robeson, Harold B., Nazareth Cement Co., Nazareth, Pa.
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Robbins, Henry C. (See Densmore LeClear & Robbins)
*Robinson, A. P. (See Ransome Concrete Mach. Co.)
Robinson, G. G. (See Standard Paving & Materials Ltd.)
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Rockwood, Nathan C. (See Rock Products)
Rodes, A. B. (See Franklin Limestone Co.)
Rogers, Eben, c/o Alton Brick Co., Alton, Ill.
Rodrigvez, Lvis M., 19 Y L Num. 172, Vedado, Havana, Cuba
Roemer, Edw. W. (See Boston Building Dept.)
*Roeth, A. C. (See Inland Steel Co.)
Rogan, W. B., 1857 East 82nd St., Cleveland, Ohio
Rogers Co., J. S., 859 Drexel Bldg., Philadelphia, Pa. (J. Clyde Rogers)
Rogers, J. Clyde (See J. S. Rogers Co.)
Rolfsen, O., Notodden, Norway
Rooney, Geo. T. (See The Scully Co.)
***Roos Co., H. W.,** Cincinnati, Ohio (H. W. Roos, Pres.)
*Roos, H. W. (See H. W. Roos Co.)
Roos Co., The H. W., 2036-46 Dana Ave., Cincinnati, Ohio (H. W. Roos, Pres. & Treas.)
Roos-Meyer-Hecht Co., The, 2814 Stanton Ave., Cincinnati, Ohio (Geo. W. Meyers, Secy-Treas.)
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Rosser, Robert C., 296 Madison St., Wilkes Barre, Pa.
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Rowe, E. D. (See Canton Block & Tile Co.)
Rowland, Walter, c/o Atmospheric Nitrogen Corp., Hopewell, Va.
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Ryan, Joseph J. (See Ryan Ready Mixed Concrete Corp.)
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Rylander, Paul N., V. P. Macdonald Spencer Engineering Co., 420 Lexington Ave., New York, N. Y.
Ryley, E. G. (See Trussed Concrete Steel Co. of Canada)
Saeger, Geoff. A., Missouri Portland Cement Co., 1010 Pine St., St. Louis, Mo.
Ste. Amedes Chaux et Ciments de La Farge et du Teil, La Teil, Ardeche, France (M. Rengade)
St. Joseph Cinder Block Co., St. Joseph, Mo. (Robert Brown)
St. Louis Concrete Products Mfg. Co., 957 North & South Rd., Brentwood, Mo. (Frank J. Grutsch)
St. Louis Duntile Co., Route No. 5, Webster Groves, Mo. (Joseph Amend, Secy.)
St. Louis Material & Supply Co., 314 N. 4th St., St. Louis, Mo. (Otto S. Conrades)
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Sanderson & Porter, 52 William St., New York City
Sandstone Quarries Co., The, 20 Groveland Ave., Minneapolis, Minn. (C. D. Lynds)
Sanford, L. M., 400 Hiram Sibley Bldg., Rochester, N. Y.
San Joaquin Light & Power Corp., Fresno, Calif. (J. M. Buswell)
San Juan, Department of the Interior, San Juan, Porto Rico) Guillermo Esteves)
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Sarwar, M. G., Punjab P. W. D., Hydro Electric Branch, P. O. Barot, via Jogindar Nagar, (Mandi State) India

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Scheidenhelm, Edward L. (See Edward L. Scheidenhelm Co.)
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Schmitt, Eugene H. (See H. Schmitt & Son Inc.)
Schmitt & Son Inc., H., 930 E. Burleigh St., Milwaukee, Wis. (Eugene H. Schmitt)
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Scott, J. R., We'land, Ont., Canada
Scott, W. M. (See Greater Winnipeg Water District)
*Scott, W. W. (See Laclede Steel Co.)
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Senior, John L. (See Consolidated Cement Corp.)
Sensing, W. C. (See Wilson-Weesner-Wilkinson Co.)
Seron, Levon, 11 Inman St., Cambridge, Mass.
Servey, Dan F. (See The Haydite Co.)
Setna, J. N., c/o Messrs. Thomas Cook & Sons, Bombay, India
Severin Co., N. P., 222 W. Adams St., Chicago, Ill. (B. J. Myren)
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Sharman, Basil C. (See John Ellis & Sons, Ltd.)
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Shedd, George G. (See L. H. Shattuck, Inc.)

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Shelverton, W. L., Washington, Ga.
Shenk, A. B. (See Federal American Cement Tile Co.)
Shenk, A. M. (See McEverlast, Inc.)
Shenk, E. R. (See Henry Shenk Co.)
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Sherrerd, Francis H. (See Newark Concrete Pipe Co.)
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Siesel, S. M. (See S. M. Siesel Co.)
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Simpson, Jr., Alex. (See Alex. Simpson, Jr., Co.)
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Simpson, Geo., 250 Park Ave., New York, N. Y.
Simpson Co., W. E., 412 Nat'l Bank of Com. Bldg., San Antonio, Texas (W. E. Simpson)
Simpson, W. E. (See W. E. Simpson Co.)
Sinclair & Grigg, 1518 Walnut St., Philadelphia, Pa. (Harris Britt)
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Sleeman, H. F. (See The R. Thomas & Sons Co.)
Sleight, T. R., Toledo Concrete Pipe Co., Silica, Ohio
Smaill, Wm. (See Northern Construction Co., Ltd.)
Small, A. R. (See Underwriters' Laboratories)
Small, H. W. (See Arnold Construction Co.)
Smallhorn, E. R., The Aerocerte Construction Co., Basin & Seigneurs Sts., Montreal, Que., Canada
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Smidth & Co., F. L., 225 Broadway, New Nork, N. Y. (O. E. Mogensen, Sec.)
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Smith, Blaine S. (See Pennsylvania Dixie Cement Corp.
Smith & Brown, 307 N. Michigan Ave., Chicago, Ill. (Albert Smith)
Smith, Geo. A. (See Johns-Manville Corp.)
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Societe Des Ciments Francais, 80 Rue Taitbout, Paris, France (F. Turquais)
Soga, Susumu, c/o The Okura Doboku Kabushikikaishia, Ginza Tokyo, Japan
Soc. Porcheddu Ing. G. A., Cors Valentino 20, Torino, Italy, (G. A. Porcheddu)
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Solvay Process Co., The, Syracuse, N. Y.
Sorenson & Sons, Julius, 21st & Northshore Tracks, Racine, Wis.
Soule, Edward L. (See Soule Steel Co.)
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South Carolina St. Highway Dept., Columbia, S. Car. (N. S. Anderson, Maint. Engr.)
South Carolina Highway Dept., Columbia, S. C. (W. H. Mills, Jr., Test. Engr.)
Southern Cement Products Co., Knoxville, Tenn. (Fred W. Weigel)
South Dakota Cement Plant, Rapid City, So. Dak. (Wm. Fowden, Supt.)
South Dakota Concrete Products Co., Box 196, Watertown, S. Dak. (Hugh H. Johnston, Mgr.)
Southern California Edison Co., Edison Bldg., Los Angeles, Calif.
Southern Cement Products Co., 521 Menchaca St., San Antonio, Texas (Harvey Perrin)
Southern Kraft Corp., Panama City, Pa. (Erling Riis, Chief Engr.)
Southwestern Engineering Co., 306 McDaniel Bldg., Springfield, Mo. (L. D. Rosenbauer)
Southwestern Laboratories, Box 1008,, Ft. Worth, Texas (F. B. Porter, Pres.)

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Standards Association of Australia, Science House, Gloucester & Essex Sts., Sydney, N. S. W., Australia (W. R. Hebblewhite, Gen. Sec.)
Standard Clay Products Ltd., St. Johns, Quebec, Canada (C. T. Trotter)
Standard Concrete Pipe & Curbing Co., 701 S. Tonti St., New Orleans, La.
Standard Gravel Co., New Hudson, Mich. (O. E. Gooding)
Standard Inspection Co., 119 Federal St., Pittsburgh, Pa. (John M. Bailey)
Standard Oil Company, Library, 225 Bush St., San Francisco, Calif. (Margaret Hatch, Librarian)
Standard Oil Development Co., (Rayway) P. O. Box 37, Elizabeth, N. J. (Chas. E. Haupt)
Standard Paving & Materials Ltd., 402 Harbor Bldg., Toronto, Ont., Canada (G. G. Robinson)
Standard Portland Cement Co., P. O. Box G, Painesville, Ohio (Erie J. Ochs, Supt.)
Standard Slag Co., Youngstown, O. (F. E. Miller, Sales Mgr.)
Stanford University, Stanford University, Calif. (Nathan Van Patten, Director)
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Stewart, L. C. (See the Dow Chemical Co.)
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Stineman, Norman M. (See Concrete Publishing Co.)
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Stuart, J. H., Stuart Vault Works, Bremen, Ohio
Stuart, Percy R. (See Calif. Stucco Products Co. of N. C.)
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Sullivan, P. L., Escanaba, Mich.
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Super Concrete Emulsions, Ltd., 3710 So. Alameda St., Los Angeles, Calif. (C. E. Little)
Superior Portland Cement Co., Concrete, Wash. (C. L. Wagner, Supt.)
*__Superior Portland Cement Co.,__ Seaboard Bldg., Seattle, Wash. (E. P. Lucas)
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Taylor, Dean T. U. (See University of Texas)
Taylor & Co., Richard, 331 Collins St., Melbourne, Victoria, Australia (Frank J. Hannan)
Taylor, S. V. (See Concrete Steel Fireproofing Co.
Teaze, Moses Hay, 31 Clarendon Place, Bloomfield, N. J.
Technische Hoogeschool, Oostplantsoen, Delft, Holland (J. A. Bakker)
Teknisk Bibliotek, Polytecknisk Laereanstalt, Kopenhagen K, Denmark (Helge Holst, Librarian)
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Thomas, Ernest (See Ernest Thomas Cast Stone Co.)
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Thomas & Sons Co., The R., Lisbon, Ohio (H. P. Sleeman)
Thompson & Binger, 565 Fifth Ave., New York, N. Y. (W. D. Binger)
Thompson, Carroll R., Dept. Wharves, Docks, Ferries, Chestnut St. Pier, Philadelphia, Pa.
Thompson, Dewey (See H. C. Nutting Co. of N. J.)
Thompson, G. Saxton, 1549 Lee Road, Cleveland Heights, Ohio
Thompson, James, 140 Cedar St., New York, N. Y.
Thompson, John H., Sterling Road, Fenimore Park, Harrison, N. Y.
Thompson-Lichtner Co. Inc., Statler Bldg., Boston, Mass. (Miles N. Clair)
Thompson-Lichtner Co., Inc., Statler Bldg., Boston, Mass. (Sanford E. Thompson)
Thompson, R. M., Imperial Bk. Bldg., Sandwich, Ont., Canada
Thompson, S. E. (See Thompson Lichtner Co.)
*Thompson, Scott (See Lone Star Cement Co.)

*Thompson-Starrett Co., 250 Park Ave., New York, N. Y.
Thompson, Warren H., 616 Stock Exchange Bldg., Buffalo, N. Y.
Thompson, W. N., c/o H. K. Ferguson Co., Hanna Bldg., Cleveland, O.
Thomson, Duncan, Onondaga Litholite Co., 102 N. Beech St., Syracuse, N. Y.
Thomson, Gordon, Experimental Engineering, University of Minnesota, Minneapolis, Minn.
Thomson, H. F. (See General Material Co.)
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Thurston, W. P. (See W. P. Thurston Co., Inc.)
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Tillson, H. L. (See American Vibrolithic Corp.)
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*Toch, Maximillian (See Toch Brothers, Inc.)
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Transit Mixers, Inc., 430 Call Bldg., San Francisco, Calif (E. T. Tuller, V. P.)
Traver Supply Co., W. G., 1902 N. Water St., Decatur, Ill. (W. G. Traver)
Traver, W. G. (See W. G. Traver Supply Co.)
Treanor, John, c/o Riverside Cement Co., 621 S. Hope St., Los Angeles, Calif.
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Trickel, J. W. (See Aldrich Construction Co.)
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*Turner Construction Co., 420 Lexington Ave., New York, N. Y. (E. J. Moore)
*Turner Construction Co., 420 Lexington Ave., New York, N. Y. (H. C. Turner)
*Turner Construction Co., Architects' Building, Philadelphia, Pa. (J. A. Turner)
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Union Pacific System, 1416 Dodge St., Omaha, Nebr. (H. C. Mann)
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Universal Atlas Cement Co., 632 Brown Marx Bldg., Birmingham, Ala. (J. C. Meacham)

*Universal Atlas Cement Co., Chrysler Bldg., 135 East 42nd St., New York, N. Y. (E. K. Borchard)
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*Universal Atlas Cement Co., 210 So. La-Salle St., Chicago, Ill.
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Warren, Ray V. (See Western Pennsylvania Sand & Gravel Assn.)
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Watson, W. P. (See The Hamilton Gravel Co.)
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Weeber, E. R. (See Hamilton & Weeber)
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Western Pennsylvania Sand & Gravel Assn., 409 Empire Bldg., Pittsburgh, Pa. (Ray V. Warren)
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Wolverine Portland Cement Co., Coldwater, Mich.
Wood, David B., c/o Alcoa Power Co. Ltd., Kenogami, Que., Canada
Wood, Douglas, U. S. Bureau of Reclamation, Las Vegas, Nev.
Wood, J. D. (See Idaho Department of Public Works)
Wood, James, 150 W. 179th St., Bronx, N. Y.
Wood, J. V., 110 Spencer St., Melbourne, Vict., Australia
Wood L. H. (See Central Texas Iron Works)
Woodard, Silas H., 10 East 40th St., New York, N. Y.
Woodle, Jr., A. S., The Baldwin Locomotive Works, Philadelphia, Pa.
Woodruff, G. B., Overbrook Lane, Darien, Conn.
Woods, Hubert, c/o Riverside Cement Co., Riverside, Calif.
Woodstock Slag Corp., Southern Railway Bldg., Birmingham, Ala. (Geo. A. Mattison, Jr.)
Woody, C. C. (See Madison Silo Co.)
Worcester, J. R., 79 Milk St., Boston, Mass.

Worthen, A. L. (See The Connecticut Quarries Co. Inc.)
Wortman, Dietrich, 116 Lexington Ave., New York, N. Y.
Wright, A. H. W., Branch Mgr. & Engr., Palmerston, N., New Zealand
Wright, Chester C., 604 West 9th St., Austin, Texas
Wright, Harold B., 33 West Grand Ave., Chicago, Ill.
Wright & Kremers, Inc., Box 435, Niagara Falls, N. Y. (Ernest Kremers)
Wrzesien, Waclaw, ul. Nowowiejska 8 m 10, Warszawa, Poland
Wu, Pond S., c/o Chief Inspectorate of Salt Revenue, No. 18, The Bund, Shanghai, China
Wurster Construction Co., 307 Architects Bldg., Los Angeles, Calif. (V. L. Marsh, V. P.)
Wyeth-Scott Co., The, Manning St. & B. O. Railroad, Newark, Ohio (W. D. Wyeth, Pres.)
Wyatt, W. Kirk, Reading Terminal, Philadelphia, Pa.
Wyatt, Roy M. (See Horner & Wyatt)
Wyeth, W. D. (See The Wyeth-Scott Co.)
Yakima Cement Products Co., 611 No. Front St., Yakima, Wash. (W. P. Hews)
Yamashita, Toshirow, C. 12-6 Maruno-Uchi, Tokyo, Japan
Yandell & Conger, E. Lafayette St. and M. & O. R. R., Jackson, Tenn. (C. H. Yandell)
Yandell, C. H. (See Yandell & Conger)
Yeats, Alex J., 85 Farnham St., Belmont, Mass.
Yingling, Clinton K., 3000 Conn. Ave., Apt. 229, Washington, D. C.
Yosemite Portland Cement Corp., Drawer "W," Merced, Calif. (G. A. Fisher)
Young, Charles H., 600 Builders Exchange Bldg., Minneapolis, Minn.
Young, Clarence Richard, 98 Hilton Ave., Toronto, 10, Ontario
Young, Fred W., Box 46, Center Square, Pa.
Young, R. B., 8 Strachan Ave., Toronto, Ont.
Young, Roy N., Young Bldg., Allentown, Pa.
Young, W. J., Ch. Engr. The Standard Lime & Stone Co., P. O. Box 785, Martinsburg, W. Va.
Younghusband, V. G. (See The Foundation Co. of Canada Ltd.)
Youngstown Burial Vault Co., 546 Indianola Ave. E., Youngstown, Ohio (E. E. Kling, Mgr.)
Youngstown Sheet & Tube Co., The, East Chicago, Ind. (C. M. Bradley)
Ytterberg, C. F., 110 E. 42nd St., New York, N. Y.
Yu, M. T., Rue Verdun and Rue Takou, Tientsin, China
Zabriskie, W. F. (See Gabriel Steel Co.)
Zachry, H. B., Engineering & Constr. Co., Laredo, Texas
Zagoren, Louis I. (See The Goldsmith Metal Lath Co.)
Zass, W. W., c/o State Highway Commission, Little Rock, Ark.
Zaugg, F. R. (See Northwest Concrete Products Assn.)
Zehrlaut, H. A. R. (See Plastic Stone Products Corp.)
Zeidler, John L., 401 Francis St., St. Joseph, Mo.
Zeidler Concrete Products Mach. Co., Waterloo, Iowa (A. W. Schultz)
Zeiget, Jr., Conrad (See Baltimore Bureau of Standards)
Zipprodt, Roy R., 347 Madison Ave., New York, N. Y.

Geographical List — A. C. I. Membership

(For detailed addresses see Alphabetical List)

Alabama

Auburn—Barrow, A. C.
Birmingham — Alabama Concrete Pipe Co.
Birmingham Public Library. (Lila May Chapman, Director)
Birmingham Slag Co.
Cement Products Co. (G. L. Cox, Pres.)
Clark, W. B.
Connors Steel Co. (Geo. W. Connors)
*Ford, Bacon & Davis, Inc. (Geo. I. Rhodes, Vice-Pres.)
*Lone Star Cement Co. (J. W. Johnston)
Merrill, B. S.
Universal Atlas Cement Co. (J. C. Meacham)
Woodstock Slag Corp. (Geo. A. Mattison, Jr.)
Mobile—Converse & Co., J. B.
Muths, George A.
Montgomery — Houck, Howard H.
Tuskegee Inst.—Gibson, Jr., Chas. H.

Arizona

Phoenix — Arizona Highway Dept. (J. W. Powers)
Tucson—Headman, Sasha

Arkansas

Fayetteville—Spencer, W. R.
Little Rock—Lander, R. S.
Zass, W. W.
Magnolia—Hardy, A. W.

California

Alhambra — MacGlashan Associates, The (W. F. MacGlashan)
Azusa—Carlson, Roy W.
Berkeley — Davis, Harmer E.
Davis, Raymond E.
Rice, John A.
Thacker, G. Q.
Tibbets, Sydney A.
Troxell, G. E.
Claremont — Edwards, Harlan H.
Colton—*California Portland Cement Co. (W. C. Hanna, Chief Chemist)
Davenport — Santa Cruz Portland Cement Co. (E. W. Rice)
Emeryville — Macintyre, A. D.
Thomas, Harry E.
Fresno — Jourdan, Concrete Pipe Co. (H. W. Chutter)
McKenzie, A. H.
San Joaquin Light & Power Corp., Library. (J. M. Buswell)
Twining, F. E.
Gilroy—Beckwith, G. L.
Long Beach — Sherer, Horace C.
Los Angeles — Allen, George F.
Architecture and Allied Arts, Library of
Baruch Corp., Herbert M. (Herbert M. Baruch, Pres.)
Beckett, Garner A.
Bent Bros. (Arthur S. Bent)
Bent, Ernest F.
Binder, R. W.
Clay Products Institute of Calif.
Cline, L. A.
Eaton, E. C.
Emme, Frank E.
Jeffers, Paul E.
Los Angeles City Engineering Dept. (T. A. Fitch, Engr.)
Los Angeles Dept. of Water and Power. (E. F. Scattergood)
Los Angeles Public Library. (Albert C. Read)
Los Angeles Public Library. (Albert C. Read)
Los Angeles Testing Labratory. (H. H. McCall)
McEverlast, Inc. (A. M. Shenk, Pres.)
Materials, Inc. (E. C. Pohlman)
Meyer & Holler, Inc. (R. Howard Armin)
Noetzli, Fred A.
Noice, Blaine
Osborne, Raymond G.
Pacific Coast Talc Co. (Geo. Ames, Mgr.)
Phillips, Wm. B.
Potts, A. H.
Powell, F. H.
Quinton Code & Hill — Leeds & Barnard (Louis C. Hill)
*Riverside Cement Co. (R. C. Aussieker)
Southern California Edison Co.
*Southwestern Portland Cement Co. (F. H. Powell)
Struthers, O. C.
Super Concrete Emulsions, Ltd. (C. E. Little)
Treanor, John
Trueblood, Wilford S.
Tuthill, Lewis H.
Wurster Construction Co. (V. L. Marsh, V. P.)
Wheelon, E. R.
Wig, R. J.
Wurster Construction Co. (V. L. Marsh, V. P.)
Merced—Yosemite Portland Cement Corp. (G. A. Fisher)
Monterey—Hoffman, R. B.
Oakland—Davis, A. P.
Dyer Construction Co., The (Kenneth W. Dyer)
Hanna, Frank W.
Howland, M. R.
Hutchinson, Jr., Hardye
Kaiser Paving Co. (T. M. Price)
Phillips, W. H.
Schorer, Herman
Pasadena — Beman, Willard J.
California Institute of Technology. (R. R. Martel, Assoc. Prof.)
Pasadena, City of
Pasadena Water Dept. (S. B. Morris)
Piedmont, Oakland — Hammill, Harold B.
Riverside — MacDonald, Earle
Woods, Hubert
Sacramento — State of California, Div. of Highways. (T. E. Stanton)
Hawley, Geo. W.
Perkins, Wayne A.
San Bernardino — Glen, Michael G.
San Diego—Jewett, John Y.
King, Thomas H.
Williams, Charles P.
San Francisco—Barrett & Hilp. (J. F. Barrett)
Brunnier, H. J.
Calif. Stucco Products Co. of N. C. (Percy R. Stuart)
Cowell Lime & Cement Co.
Day, W. P.
Denton, Arthur P.
Dewell, Henry D.
Dreyer, Walter
Ellison & Russell (W. H. Ellison)
Furlong, Irving
Grassi & Co., P. (P. Grassi)
Gunn, Charles M.
Hanks, Inc., Abbot A. (Theo. P. Dresser, Jr., Chief Engr.)
Harding, Carroll R.
Huber, Walter L.
Jellick, J. E.
Jorgenson, Lars
Kennoy, John S.
Leonard, John B.
McKesson, Claude L.

Nishkian, L. H.
*Santa Cruz Portland Cement Co. (George R. Gay)
Saph, Augustus V.
Soule Steel Co. (Edward L. Soule)
Spaulding, Horace R.
Standard Oil Co. Library. (Margaret Hatch, Librarian)
Stephens, Carroll
Transit Mixers, Inc. (E. T. Tuller, Vice-Pres.)
Villadsen, Jens. M.
San Marino — Kressly, Paul E.
San Mateo—Deuel, A. W.
Santa Monica—Collings Co., W. A. (W. A. Collings, Mgr.)
San Simeon—Hayden, J. M.
Stanford University — Stanford University (Nathan Van Patten)
Thomas, Earl C.
Stockton—Burke, J. M.
Watsonville — Granite Rock Co. (R. E. Fowle)
Wilmington—Los Angeles Harbor Dept. (G. F. Nicholson)

Colorado

Boulder—Gilkey, Herbert J.
Raeder, Warren
Denver—Cox, O. H.
Gowdy, Roy C.
Haring, Ellis E.
Kirchhof Constr. Co., F. J. (F. J. Kirchhof)
Meade, P. F.
Northwestern Terra Cotta Co., The. (Geo. W. Mueller)
Pierce Testing Laboratories, The. (George Pierce)
Ruettgers, Arthur
Savage, J. L.
Shankland, Ristine & Co. (R. G. Shankland)
Simpson, Jr., Co., Alex. (Alex. Simpson, Jr.)
Steele, Byram W.
Stiefel, Alfred C.
Weber, Walter R.
Williams, Jay T.
Fort Collins—Road Materials Testing Laboratory. (C. A. Carpenter)
Julesburg—Parker, Philip M.

Connecticut

Bridgeport — Bridgeport Stone Co., The. (Frank E. Clarke)
Fletcher - Thompson, Inc. (Edward A. Lambert)
Clinton — Chittenden, Howard L.
Darien—Woodruff, G. B.
Hamden — Dextone Co., The (Louis A. Falco)
West Hartford—Crosby, Henry L.
Poriss Co., The S. C. (S. C. Poriss)
Hartford—Balf Company, The Edward
Ennis, Wm. J.
Hartford, City of (Philip A. Mason, Bldg. Supervisor)
Linnell, Frank A.
Mylchreest, G. L.
Windsor Cement Co., Inc.
New Haven—Blakeslee & Sons, Inc., C. W. (W. T. Gilbert)
Connecticut Quarries Co., Inc., The (A. L. Worthen)
Crane, Theodore
Gillon, Frank
Roberts, H. W.
Portland — Conn. State Highway Dept. (F. G. Flood)
Stamford — American Sewage Disposal Co., Inc. (F. M. Brown, Pres.)
Stratford—Lake, Simon
Westbrook—Morrill, Geo. P.

Delaware

Dover — Delaware State Highway Dept. (W. W. Mack, Ch. Engr.)
Middletown—Reaves, G. M.
Wilmington—Art Stone & Tile Co. (Arthur Pavoni)

District of Columbia

Washington — Allen, T. Warren
Bates, P. H.
Bogue, Robert H.
Bradbury, R. D.
Burggraf, Fred
Clemmer, H. F.
Crum, R. W.
Davis, Watson
Denton, W. Edward
Drew Co., Inc., Fred. (Fred Drew)
Dwyer, John R.
Earley, John J.
Edwards, Llewellyn N.
Fishburn, Cyrus C.
Ginder, J. W.
Ingberg, S. H.
Jackson, F. H.
Lapham, John R.
McDaniel, Allen B.
Marsh, Earl G.
Marshall, Thos. W.
Morgan, Nathan W.
National Crushed Stone Association. (A. T. Goldbeck)
National Lime Association. (W. V. Brumbaugh)
National Paving Brick Mfgs. Association (F. G. Schlesinger)
Oehmann, John W.
Parsons, Douglas E.
Proudley, C. E.
Rapp, Paul
Raywid, Leo.
Rich, Melvin S.
Ross, Culbertson W.
Rossman, Joseph
Ruebsam, Ernest C.
Saum, Irving R.
Schuman, Louis
Tompkins, Charles H.
Toner, A. C.
Trainor, Lee S.
Tucker, Jr., John.
Upham, C. M.
Walker, Stanton.
Wells, Hewitt
Yingling, Clinton K.

Florida

Daytona Beach — Florida Concrete Products Assn. (Walter Snead, Secy.)
Gainesville — Driggers, Clyde
Florida State Road Dept. (H. C. Weathers)
University of Florida.
Melbourne — VorKeller, Albert
Miami—Hannock, Charles F.
West Palm Beach — Chace, Clyde B.
Panama City—Southern Kraft Corp. (Erling Riis)
Tampa—Florida Portland Cement Co. (Cyrus N. Wells)

Georgia

Atlanta — American Memorial Co. (T. J. Meldrum)
Harris, Robert Olin
Lose, Robt. G.
McCrary Engineering Corp., J. B. (E. C. Catts, Purch. Agt.)
Marshall, Jr., Jos. M.
Oliver, Herbert D.
Spiker & Co., W. C. (William C. Spiker)
Willingham - Little Stone Co. (H. S. Willingham, Pres.)
Willson, S. H.
East Point—Slack, Searcy B.
Macon—Elkan, Stanley A.
Savannah — Cut Art Stone Co. (M. E. Kleinsteuber)
Washington—Shelverton, W. L.

Idaho

Boise — Department of Public Works. (J. D. Wood)
Wiley, A. J.
Pocatalleo—Althof, L. W.
Priest River—Robertson, R. R.

Illinois

Alton—Rodgers, Eben
Arlington Heights—Arlington Concrete Products Co. (Paul C. Taege)
Champaign—Johnson Co., C. S. (Chas. S. Johnson)
Nelsen Concrete Culvert Co., Inc. (E. P. Shapland)
Oliver, William A.
Chicago—Affleck, B. F.
Aldrich Construction Co. (J. W. Trickel, Pres.)
Allan, W. D. M.
Allen, O. T.
American Concrete Marbelite Co. (Paul K. Fleming)
American Concrete Pipe Association. (M. W. Loving)
American System of Reinforcing. (J. T. Hanley)
Anderson, Arvid A.
Architectural Stone Co. (Nelson A. Friis)
Associated Metal Lath Mfrs. (Wharton Clay)
Babcock, E. P.
Bartlett, G. S.
Battey & Kipp, Inc. (P. L. Battey)
Baylis, John R.
Bedford Construction Co. (N. H. McKenzie)
Bennett, J. Gardner
Benson Concrete Co. (C. H. Johnson)
Brown, J. F.
Brundage, Avery
Bullen, Carroll A.
Burrell Engineering & Construction Co. (Joseph Wilson, Secy.)
*Calumet Steel Co. (F. G. Carrol)
Capouch, M. E.
Cement Gun Construction Co. (John V. Schaefer)
Chicago Art Marble Co. (M. S. Marcus, Pres.)
Colburn, D. S.
Colorway, Inc. (Marshall R. Rust)
Concrete Publishing Co. (Norman M. Stineman)
Concrete Reinforcing Steel Institute. (R. W. Johnson)
Condron & Post. (C. L. Post)
Consolidated Cement Corp. (A. W. King)
Cummings, A. E.
Curtis, Benjamin J.
Cutler, Stanley G.
Davis, E. E.
DeLeuw & Co., Charles E.
Dillingham, George M.
Doucha, J. C.
Electric Tamper & Equip Co. (M. I. McCarty)
Emerson, H. B.
Engel, W. L.
Epstein, A.
Erickson, Ralph E.
Ericsson Co., Henry
Farmer, Homer G.
Federal American Cement Tile Co. (A. B. Shenk)
Floto, Julius
Foster, Winslow H.
Fridstein, Meyer
Gardner, Franc E.
Gonnerman, H. F.
Hart, W. E.
Harza L. F.
Hoeffer & Co., (Alexander C. Warren)
Holabird & Root. (E. A. Renwick)
Humboldt Mfg. Co. (Louis H. Martens, Pres.)
Hyde, George A.
Illinois Steel Co. (E. B. Harkness)
Ingemanson, Thure W.
Inland Steel Co. (A. C. Roeth)
Johnson, A. W.
Johnson, Otis Floyd
Kalinka, J. E.
Kalman Floor Co., The. (C. E. Cooke)
*Kalman Steel Co.
Keatinge, Paul F.
Kinney, William M.
Krause, L. B.
Langdon, E. W.
Langeler, M. Bert
Larson, D. E.
Leffler, Ralph R.
Lieberman & Hein. (E. Lieberman)
Lindau, A. E.
Lindsay, G. L.
Logeman, R. T.
Lord, Arthur R.
Lowell, J. W.
Lubin, Frank
Lynch, W. J.
McCaully, W. H.
McKay, Richard T.
McKeague, E. D.
McKinstry, Ross W.
McMillan, Franklin R.
Mack, Thomas
*Marquette Cement Manufacturing Co. (R. R. Coghlan, Chem. Engr.)
Martin, Edgar.
Masonite Corporation (E. B. Bushnell)
*Massey Concrete Products Corp. (E. C. Alexander)
Moore, O. L.
Moulton, A. G.
Oesterblom, I.
Pearse, Langdon
Pease, B. S.
Perfected Burial Vault & Cement Products Co. (H. A. Dahlquist, V. P.)
Perry, J. P. H.
Pettersen, Thomas
Phillips, R. S.
Powers, Treval C.
Rader, B. H.
Rail Steel Bar Association. (H. P. Bigler)
Randall, Frank A.
Ritter, Louis E.
Roberts & Schaefer Co. (E. E. Barrett, Pres.)
Rock Products. (Nathan C. Rockwood, Editor)
Rush, D. B.
Sargent & Lundy, Inc. (J. C. Sanderson)
Sauermann, Otto
Scheidenhelm Co., Edward L. (Edward L. Scheidenhelm)
Schuettge, Ralph T.
Schwantes, E. J.
Seidensticker, F. W.
Severin Co., N. P.
Shapiro, Benj. B.
Shideler, J. Raymond
Sisalkraft Co., The. (E. T. Anderson, Pres.)
Smith & Brown. (Albert Smith)
Spooner, Chas. W.
Stern, I. F.
Stone Brick & Block Works, Inc. (Paul E. Scott, Pres.)
Strauss, Joseph B.
Suhr, Berryman Peterson & Suhr. (Ivan C. Peterson)
*Universal Atlas Cement Co. (B. F. Affleck, Pres.)
*Universal Atlas Cement Co.
Van Houten, Chas.
Van Trump, Isaac
Voshell, James T.
Walker, C. G.
Walker, Frank R.
Wallace, Robert W.
Warren, George E.
Weber, Lester N.
Whitacre Engineering Co. (Arthur Hagener)
White, William E.
Wilson, Raymond
Wilson & Co., R. F. (R. F. Wilson)
Winokur, M. C.
Witt, J. C.
Wright, Harold B.
Chillicothe — Weidman, H. P.
Danville—Dierstein, A. L.
Western Brick Co. (E. T. Strawbridge)
Decatur—Traver Supply Co., W. G. (W. G. Traver)
Des Plaines—Des Plaines Concrete Products Co. (Chas. Gatzke)
Downers Grove—William M. Labastille
Pyramid Company (W. H. Frazier)
East St. Louis—Gain, Elmer W.
Mepham & Co., Geo. S. (L. K. Ayers, V. P.)
Elmhurst — Elmhurst-Chicago Stone Co. (Martin Hammerschmidt)
Evanston—Coy, Frank A.
City of Evanston. (E. M. Goodman)
Maney, G. A.
Nielsen, Frederick E.
Shields, H. C.
Glen Ellyn—Clarahan, Jr., Charles H.
Granite City — Hilker Supply Co. (E. W. Hilker)
Hinsdale—Brock, Arthur S.
Joliet — Ideal Concrete Co. (John G. Segenberger)

Ideal Concrete Co. (John G. Segenberger)
LaGrange—Ramm Co., E. L. (E. L. Ramm)
La Salle—Marquette Cement Mfg. Co. (C. M. Butler, Chief Chemist)
Mattoon—Goetz, John A.
Maywood—Chicago Architectural Stone Co. (M. A. Williams)
Moline — Axel-Carlson Co. (Curt Lundeen)
Oak Park—Concrete Bldg. Units Co. (P. F. Huxhold, Secy.)
Ottawa — Ottawa Silica Co. (P. S. McDougall, General Manager)
Peoria—Hill, W. N.
Hoerr, Philipp J.
Waterman, L. R.
Quincy—Buerkin & Buerkin. (Julius A. Buerkin)
Springfield — Illinois Division of Highways, Bureau of Materials.
Russell, Howard W.
Smith Chemical Works, Inc., Geo. B. (R. R. McCoy)
Stockton—Klassy. K. H.
Urbana—Bauer, E. E.
Brown, Rex L.
Crandell, John S.
Cross, Prof. Hardy
Foster, W. A.
Huntington, W. C.
Ketchum, M. S.
Lansford, Wallace M.
Morgan, N. D.
Murphy, Glenn
Newcomb, Rexford
Putnam, Wm.
Richart, F. E.
Roy, N. H.
Schwalbe, William
Talbot, Prof. Arthur N.
Tomita, M.
Westergaard, Harold M.
Winnetka — Windes & Marsh. (Frank A. Windes)
Winnetka, Village of (Robt. L. Anderson)
Wood River — Kienstra Bros. Fuel & Supply Co. (Frank T. Kienstra)

Indiana

Bedford—Huber, Lee
East Chicago — Youngstown Sheet & Tube Co., The. (C. M. Bradley)
Hammond—Federal Cement Tile Co. (J. George Geyer, Gen. Supt.)
Indianapolis — *Baker & Co., Hugh J. (Hugh J. Baker)
Geupel, Carl M.
Hall Construction Co.
Harrison, Merritt
Independent Concrete Pipe Co. (R. D. Robertson)
*Indiana Portland Cement Co. (Geo. E. Pierson, Mgr.)
*Insley Mfg. Co. (Alvin C. Rasmussen)
Kellam, Fred
Miesenhelder, P. D.
Miller, L. C.
Ratcliff, T. R.
Stackhouse, A. V.
W. Lafayette—Crepps, R. B.
Springer, G. P.
Lafayette—Freel, W. I.
Hatt, William K.
Hollister, S. C.
Kemmer, A. E.
Peru—Automatic Sealing Vault Co., The (C. C. Millhouse)
Richmond — Mueller, J. W.
Wabash—Philpott, James H.

Iowa

Ames—Morris, Mark
Stoddard, A. E.
Bettendorf —Bettendorf Co., The. (R. M. Bowling)
Buffalo—Breitengross, R. A.
Cedar Rapids — Cinder Block & Material Co. (H. L. Spaight)
Green, Howard R.
Davenport — Crabbs, Austin
Dewey Portland Cement Co. (K. P. Ferrell)
Priester Construction Co. (O. F. Priester)
Des Moines — American Vibrolithic Corp. (H. L. Tillson, Chief Engr.)
Iowa Concrete Crib & Silo Co. (H. E. Kilmer, Gen. Mgr.)
Mandia, Jas. V.
Patzig, Monroe L.
Steiner, W. H.
Weitz, Fred W.
Iowa City — Hands, Stanley M.
Mavis, F. Theodore
Iowa Falls—Welden, E.
Mason City — Northwestern States Portland Cement Co. (G. C. Blackmore)
Moville—Everart, C. C.
Oskaloosa—Carlon Construction Co. (Harry F. Carlon)
Sioux City—Everist, Inc., L. G. (H. C. Boswell)
Johnson, T. H.
Klinger, W. A.
*McCracken Machinery Co. (R. M. La Due)
Waterloo—Waterloo Concrete Corp. (C. L. Douthett, Pres.)
Zeidler Concrete Products Mach. Co. (A. W. Schultz)

Kansas

Chanute — Gardner, Guy O.
Clay Center—Wheelock, D. L.
Hutchinson — Grothe, Wm. F.
Hertz, H. P.
Phillips, Leroy
Kansas City—Wilson, L. Clyde
Manhattan — Koenitzer, L. H.
Scholer, Prof. Charles H.
Topeka—Haegert, Lee V.
Wichita—Cement Stone & Supply Co. (B. F. Krehbiel, Mgr.)
Waddington, C. V.

Kentucky

Frankfort — State Highway Dept. (J. S. Watkins)
Lexington—Congleton Co. W. T. (W. T. Congleton)
Des Cognets & Co., Louis (E. R. Ackerman, Secy.-Treas.)
Terrell, D. V.
Louisville — American Builders Supply Co. (Jas. F. McCracken)
*Kosmos Portland Cement Co. (A. W. Horner)
*Louisville Cement Co. (W. S. Speed, Pres.)
Speed Scientific School. (W. B. Wendt)
St. Matthews—Russell & Co., J. F.

Louisiana

Baton Rouge — Cramer, M. J.
Kivett, Edward H.
Crowley—Bernard, Merrill
Monroe—Barr, Charles S.
New Orleans—DePass, M. B.
Derickson, Donald
Glover Co., Geo. J.
*Lone Star Portland Cement Co. (Scott Thompson)
McNiven, J. A.
Montz, A. S.
Olsen, Ole K.
Porter, John L.
Portland Cement Assoc. (W. G. Hudson)
Sawyer, H. A.
Shilstone, Herbert M.
Standard Concrete Pipe & Curbing Co.
Watkins, M. M.
Shreveport—Freeman, E. M.

Maine

Augusta — Mason, Frank H.
Biddeford—Howard, S. B.
Brunswick—Maine Concrete Co. (W. A. Jack)
Orono — Technology, Experiment Station
Portland — Goddard. Calvin S,

Maryland

Baltimore—Baltimore Bureau of Standards (Conrad Zieget, Jr.)
Bryant, Charles B.
Cinder Block Corporation (G. W. Creighton, Gen. Mgr.)
Cummins, Charles A.
Doeleman, Herman F.
Fahm, Jr., Frank
Greiner, J. E.
Kirwan, Kenneth K.
Lane, H. A.
Lang, Jr., Philip George
Meyer, A. A.
Moore, Thos. V.
Pennsylvania Water & Power Co. (N. B. Higgins, Asst. Ch. Eng.)
Saxe, Van Rensselaer P.
Wich, John F.
Chevy Chase—McBurney, J. W.
College Park—Maryland, University of (A. N. Johnson, Dean)
Cumberland—Iron City Sand and Gravel Co. (E. J. Kean)
Hagerstown — Ferguson & Co., J. B. (J. B. Ferguson)
Muirkirk—Waldo, Inc., E. M. & F. (Edward M. Waldo, Pres.)

Massachusetts

Belmont—Yeats, Alex J.
Boston — *Aberthaw Co. (L. C. Wason)
Accuvad, J. G.
Associated Factory Mutual Fire Ins. Co. (C. K. Stiff)
Barrows, Frank G.
Boston Building Dept. (Edw. W. Roemer, Commissioner)
Boston Transit Mixers, Inc. (John T. McMorrow)
Brazer, George H.
Brown, Burtis
Burke, John A.
Burns, Inc., D. F. & W. G. (Walter G. Burns)
Cinder Concrete Units Corp. (Stephen Kearney)
Colby, Edwin W.
Cutler, M. H.
Densmore, LeClear & Robbins (Henry C. Robins)
Deslauriers Column Mould Co., Inc. (Henry A. Dahlen)
Dockstader, E. A.
Dutton, Earl S.
Emerson-Norris Co. (Frederick A. Norris)
Fay, Frederick H.
Gray, Howard Allison
Grid Flat Slap Corp. (Edwin F. Allbright)
Harty, John J.
Hersum, LeRoy M.
Jackson & Moreland (E. H. Cameron)
Kerr, Linton
Linenthal, Mark
Lockwood Greene Engineers, Inc., (H. E. Cousins)
MacMillan, A. B.
MacNaughton Co., Inc., P. J. (P. J. MacNaughton)
Macomber, C. Clark
Main, Charles T.
Matthews, L. H.
Metcalf & Eddy (Frank A. Marston)
Moore, Lewis E.
Monks & Johnson (B. A. Rich)
Nichols, Charles Eliot
Nichols, John R.
Northeastern University (Henry B. Alvord)
Norton, Paul W.
Perkins, G. Howard
Reed Co., William T.
Reidy, Maurice A.
Rowe, Hartley
Sherman, Herbert L.
Stone & Webster (C. E. Nichols)
Thompson - Lichtner Co., Inc. (Miles N. Clair)
Thompson - Lichtner Co., Inc. (Sanford E. Thompson)
Towles, Thomas Thomson
Turner, J. M.
Underhill, P. H.
Worcester, J. R.
Brighton—Whelan, Frank H.
Brookline—Bryant, Henry F.
Cambridge — Adams, Ralph G.
*Boston Sand & Gravel Co. (Paul P. Bird)
California Stucco Products of N. E. (Napoleon M. Bernier)
Gow, Charles R.
Haertlein, Albert
Johnson, Lewis J.
Maeser, Mieth
Mitsch, John D.
Peabody, Jr., Dean
Scully Co., The (Geo. T. Rooney)
Seron, Levon
Swain, George F.
Tucker, Ross F.
Wilson, Francis W.
Concord — Brown, H. Whittemore
Dorchester—Dones, Samuel G.
Franz, Herbert J.
Fall River—Beattie, Roy H.
Beattie & Cornell
Jamaica Plain—Campion, William L.
Lawrence—Delaney, Wm. J.
Lenoxdale — Berkshire Gravel Co. (A. I. Newton)
Malden—Becker, Frank V.
West Medford — Allen, Cleveland
Allen, John E.
Medford—Freedman, J.
Mozzicato, James S.
Melrose—Hoyt, Glenn T.
Winchester, Frank W.
Middleborough—Trufant, Russel Alden
Millers Falls—Art Stone Co., The (I. L. Bartlett)
Natick—Carter, Wilfred E.
Newton Center—Mariner, W. Seward
Newton—Miner, Reginald B.
Sampson, George A.
Reading—Nelson, Dewey
Somerville—Hurley, John J.
Springfield—Annable, B. A.
Baker, R. K.
Clough, G. C.
Dowd, Daniel M.
Dube, Charles
Ley & Co., Inc., Fred T. (T. A. Pearson)
Roy, Joseph E.
Valentine Lumber & Supply Co. (Frank J. Murphy)
State Farm — Maynard, Arthur J.
Swampscott — Erickson, Jr., Arioch Wentworth
Waban—Mohr, H. A.
Watertown — Gallagher, John P.
Wellesley Farms—Hayes, Walter M.
Voss, Walter C.
No. Wilbraham—Birnie Sand & Gravel Co. (J. E. Bowker)
Worcester — Fiske-Carter Construction Co. (Burton C. Fiske)
Frost, Chamberlain & Edwards
Jacobson, G. W.

Michigan

Adrian — *Consolidated Concrete Machinery Co. (Eugene F. Olsen, Vice-Pres.)
Alpena—Besser Manufacturing Co. (J. H. Besser)
Ann Arbor—Ayres, Lewis, Norris & May
Emmons, W. J.
Eriksen, Edw. L.
Gram, Lewis M.
McConkey, George M.
Marshall, W. V.
Menefee, F. N.
Michigan, University of (R. L. Morrison)
Orbeck, Martin J.
Sherlock, R. H.
White, Alfred H.
Cement City—Taggart, A. D.
Coldwater — Wolverine Portland Cement Co.
Dearborn—Chamier, A. U.
Morrill, Arthur B.
Detroit — American Hume Pipe Co. (F. L. Klingensmith)
Ayres, F. C.
Bates, Harold W.
Blake, H. C.
Boomer, Roy E.
Burton, Frank
Ceglarek, Wallace
Charron, Louis
Concrete Block & Products Co. (Jos. A. Sattler)
Concrete Steel Fireproofing Co. (S. V. Taylor, Pres.)
Cooper, Gage W.
Detroit Bureau of Government Research (Harrington Place)

Detroit Cinder Block & Tile Co.
Detroit Department of Building & Safety Engr. (C. A. Daymude)
Detroit Edison Co.
Detroit Edison Co. (A. S. Douglass, Chief Engr.)
Detroit Public Library (Adam Strohm, Librn.)
Dept. of Street Railrays (P. A. Kerwin)
Detroit Testing Lab., The (W. R. DeGowin)
Detroit, University of, Library (David P. Gilmore)
Dever, Clare
Donaldson & Meier (H. W. Meier)
Elms, Stephen F.
Eyrick, Jr., George J.
Fenkell, Geo. H.
Gabriel Steel Co. (W. F. Zabriskie)
Hanneman Bros.
Hinchman, T. H.
Hoffman, L. R.
Hubbell-Hartgering & Roth (Clarence W. Hubbell)
*Huron Portland Cement Co. (J. W. Kennedy)
Kahn, Albert
Kearney, James C.
Koenig Coal & Supply Co. (A. J. O'Connor)
La Camera, Peter
Lake Ports Supply Co., The
Lenhardt, L. G.
Liskow, Fred L.
Long, J. F.
MacKenzie, W. S.
McMillan, E. C.
McRae Steel Co. (K. C. McRae)
Mason, Roscoe J.
Maul Co., The (Bruce F. Carty, Secy.)
Michigan Concrete Co. (E. H. Bingham)
Morse, Floyd C.
Outzen, Andrew N.
Perry, Ralph W.
Seeliger, Joachim Eberhart
Sexton, James E.
Stone, O. O.
Super Cement Co., The (W. J. D. Reed-Lewis)
Truscon Laboratories, The (R. A. Plumb)
Wasson, J. H.
Weaver, H. A.
Whipple, Harvey
Wilk, Benjamin
Williams, L. E.
Wolfe, W. S.

Escanaba—Sullivan, P. L.

East Lansing — Michigan State College (C. L. Allen)

Flint—Ortman, Frank A.
Robb, R. E.

Galien—Galien Concrete Tile Co. (C. A. Roberts, Secy.-Treas.)

Grand Rapids—Hamilton & Weeber (E. R. Weeber)

Lamar Pipe & Tile Co. (C. E. Edwards, Gen. Mgr.)

Jackson—Donovan, John B.
Everett, Roy E.
Stevens & Wood (Edw. M. Burd)
Willbee Concrete Products Co. (Charles E. Willbee)

Kalamazoo — Billingham & Cobb (L. A. Cobb)
Brassert, Walter
Lilly, J. A.

Lansing — Reniger Construction Co. (M. H. Reniger)
Van Eenam, Neil

Midland—Dow Chemical Co. (L. C. Stewart)

Mt. Clemens — Lehner, Walter J.

Muskegon—Lewis, H. J.

Newaygo — *Newaygo Portland Cement Co.

New Hudson—Standard Gravel Co. (O. E. Gooding)

Union City — Peerless Portland Cement Co. (William M. Hatch)

Minnesota

Duluth—Dickerson, Oliver H.
Josephs, Arthur C.
Kelly, E. W.
Pharis, L. M.
Vogt, R. C.
Polaris Concrete Products Co. (Gordon H. Butler)
Whitney Materials Co. (G. A. Whitney)

Elk River—Elk River Concrete Products Co. (D. W. Longfellow)

Minneapolis — Altman, Frank S.
Alrick, B. G.
Bergford, L. M.
Brockway, R. R.
Clark, Kenneth M.
Clousing, Louis
Fegles Construction Co., Ltd. (D. B. Fegles, Pres.)
Forrest, V. E.
Hughes, C. A.
Hustad Co., The (A. P. Hustad, Pres.)
Johnson, Algot F.
Kelley, J. W.
Lagaard, M. B.
Macgowan, Ernest S.
Minneapolis Street Ry. Co. (O. F. Moore)
Northern States Power Co. (G. E. Loughland)
Paul, Frederick T.
Sandstone Quarries Co., The (C. D. Lynds)
Straub, Dr. Lorenz G.
Thomson, Gordon
Turner, C. A. P.
Wheeler, Walter H.
White, C. J.
Young, Charles H.

St. Paul—Arnold Construction Co. (H. W. Small)
Capital Avenue Cement Works (A. O. Hovde)
Durr, C. Warren
Guaranteed Concrete Co. (A. R. Shiely, V. P.)
Hall, Quincy A.
King, W. E.
Loeffler, H. S.
Lovering-Longbotham Co. (Harry D. Lovering)
Minnesota Highway Dept. (F. C. Lang, Engr. of Tests & Insp.)
Minnesota Mining & Mfg. Co. (L. A. Hatch)
Schuett Meier Co. (A. Hs Schuett)
St. Paul Cement Work. (H. C. Berchem)

Mississippi

Meridian—P'Pool & Son, A.

Vicksburg — Birchett, Raymond

Missouri

Brentwood — St. Louis Concrete Products Mfg. Co. (Frank J. Grutsch)

Carthage—Wetzel, E. H.

Charleston—Quick, Jerry L.

Dalton—Laker Concrete Works (H. J. Laker)

Jefferson City—Missouri State Highway Dept. (F. V. Reagel, Engr.)
Remley, Norman W.

Kansas City—Anderson, W. R.
Black & Veatch (N. T. Veatch, Jr.)
Burns & McDonnell Engineering Co. (C. S. Timanus)
Cross, Roy C.
Dewey Portland Cement Co. (F. E. Tyler)
Erickson, Iver
Everham, A. C.
Haydite Co., The (Dan F. Servey)
Heidenreich, E. Lee
Jones-Hettelsater Constr. Co.
Horner & Wyatt (Roy M. Wyatt)
Hussey, Jack W.
*Kansas Portland Cement Co., The (J. A. Lehaney)
Logan, Edgar H.
Lundteigen, A.
Lyons, R. P.
McIntyre, Geo. E.
Nelson, G. V.
Nicholson, Geo. E.
Prince, John
Schramm, A. F.
Swanson, C. E.

St. Louis—Barnsdall Tripoli Co. (W. C. Bruce)
Becker, W. C. E.
Berg Vault Co. (Christy L. Berg)
Birkland, George
Colloy Production Co. (C. P. Derleth
Concrete Marble Co. (G. C. Turner)
Dailey Contracting Co., H. A.
Freuenfelder, Herman
Frech, H. E.
Friberg, Bengt.

General Material Co. (H. F. Thomson)
Hedderich, H. F.
Horner, Wesley W.
James, William
Knight, W. J.
Koerner, Carl A.
*Laclede Steel Co. (W. W. Scott)
McMahon, E. J.
Made-Rite Products Co. (Robert Fullerton, V. P).
Many, Ben J.
*Missouri Portland Cement Co. (H. C. Block, Pres.)
Parcel, John I.
Pettus, L. A.
St. LouisMaterial & Supply Co. (Otto S. Conraaes)
Saeger, Geoff A.
Sweetser, Ernest O.
Tripp, J. G.
Verschleiser, J. H.
Wheeler, W. H.
North Kansas City—Allen, H. S.
Rolla—Butler, Joe B.
St. Joseph—Leslie Engrg. Co. (J. R. Leslie)
St. Joseph Cinder Block Co. (Robert Brown)
Zeidler, John L.
Springfield — Gray, William J.
Webster Groves — St. Louis Duntile Co. (Jos. Amend, Secy.)
Southwestern Engrg. Co. (L. D. Rosenbauer)

Nebraska

Columbus — Gottschalk, L. F.
Lincoln—Abel Construction Co.
Duff, C. M.
Grone, Edwin Arthur
Sampson, B.
Louisville—Frolich, A. K.
Omaha—Curtis, L. C.
Meyer, C. Louis
Moran Construction Co., The (Frank Moran, Pres.)
Rockwell, J. A.
Swanson, A. G.
Thee, T. C.
Union Pacific System (H. C. Mann)
Wemmer, H. P.

Nevada

Las Vegas—Patch, Orin G.
Wood, Douglas

New Hampshire

Bristol—Resseguie, H. D.
Concord — Purrington, Wallace F.
Lebanon—Downs, Allan B.
Seabrook—Schroedel, F. H.
Manchester — Shattuck, Inc., L. H. (George G. Shedd)

New Jersey

Ampere—Lock Joint Pipe Co. (J. E. Longley)
Lock Joint Pipe Co. (F. F. Longley)
*Lock Joint Pipe Co. (A. M. Hirsh, Pres.)
Arlington—Brandes, Raymond L.
Atlantic City—Ferry Co. Inc., James (James V. Ferry)
Markland, M. B.
Bellville — Brooks, Robert M.
Bloomfield—Teaze, Moses Hay
Bound Brook — Branda, Christopher
Camden—Concrete Specialties Co. (L. A. Goodwin, Secy.-Treas.)
Dunellen — *Ransome Concrete Machinery Co. (A. P. Robinson)
East Orange—Albert, Odd
Atwater, Ralph W.
Christie, H. A.
Deignan, John E.
Matthews, Howard B.
Pierson Co., J. W. (James T. Pierson, Vice-Pres.)
Ulrich, Thomas J.
Elizabeth — Chapman, John J.
Standard Oil Development Co. (Rayway) (Charles E. Haupt)
Fort Lee—Bergholm, A. O.
Haddonfield — Hewes, George C.
Irvington — Core Joint Concrete Pipe Co., Inc., (G. M. Kerr)
Lindsley Co., C. E. (C. E. Lindsley)
Jamesburg—Davison, R. Glenn
Jersey City—Gilligan, William H.
Hud-Cin Building Products Inc. (W. H. Blackwood)
Mitchell, James
Munsell, A. W.
Radigan, Frank J.
Sureseal Manufacturing Co. (Walter J. Gorman, Pres.)
Walter, L. W.
Wasser, T. J.
Leonia — Morrill, Albert H.
Linden—Porter, J. Miller
Manasquan—Vogel, John Leonard
Milville — Haddow, Jr., Hugh
Moorestown—Cone, G. R.
Morristown — McCune, Stephen
Newark — Anti-Hydro Waterproofing Co. (M. W. Meyer)
Bingham, George C.
Bound Brook Crushed Stone Co. (L. Upton)
Colwell, Curtis C.
Drill, Max
Farrell, J. I.
Igoe Brothers (Ralph F. Healy)
Kindle, W. H.
Mulroy-Cooke Company, Inc. (E. J. Mulroy, Pres.)
Newark Concrete Pipe Co. (Francis H. Sherrerd, V. P.)
New Jersey Dist. Water Sp. Comm. (Arthur H. Pratt)
Nutting Co. of N. J., H.C. (Dewey Thompson)
Porete Mfg. Co.
Powers, Ellwood D.
New Brunswick—Marin, Joseph
New Village—Edison Cement Co. (George C. Wilsnack)
North Bergen — Hudson Block & Supply Co., Inc., (W. J. Dicker, Gen. Mgr.)
Old Bridge—Nowell, Jr., Joseph C.
West Orange—Devonald, Ira R.
Orange—Fossum, L. T.
Mead-Suydam Co. (F. J. Mead, Pres.)
Paterson — Anderson, Walter A.
Federici, Albert
Ferguson, A. Donald
Ferguson Co, Inc., John W. (John W. Ferguson)
Hughes, R. G.
Jones, William M.
Torrey, James E.
Plainfield—Benham, Sanford W.
Loizeaux Lumber Co., J. D. (George B. Coale)
Port Monmouth—Freibott, George B.
Princeton—Begs, George E.
Ramsey—Conklin, James Arthur
Ridgefield Park—Bergen Building Block Co. (Howard Brooke)
Ridgewood—Evans, Frank M.
River Edge—Pirkner, Herbert R.
Rutherford—Fink, John C.
Summit — Rhett, Albert H.
Teaneck—Munsell, J. R.
Trenton—Andrews, L. E.
Friebele, J. F.
Gage, Robert B.
Robbins, Harry D.
Scheirer, M. K.
Schlottenmeier, Joseph F.
Sherman, Ralph A.
Upper Montclair—Mead, C. A.

New Mexico

Albuquerque—Holm, W. M.

New York

Albany—Colmar, Daniel
Greenman, Russell S.
Kelley, Frederick W.
*Lone Star Cement Co. New York, Inc. (D. P. Cooper)

Myers, Jasper M.
Ready Mix & Supply Corp. (N. D. Crowley, V. P.)
Amsterdam — Turner & Sons, John P.
Bayside—Johnston, Robert S.
Bronx—Blancato, Virgilis
Caiola, Fred
Liebeskind, Morris Moe
Smyth, Raphael J.
Usilton, L. H.
Waterside Concrete Works (Douglas S. Thropp, Gen. Mgr.)
Wood, James
Brooklyn — Abberley. Elbert K.
American Vault Co. (F. H. VanNess)
Bekay, A. V.
Caye Construction Co., Inc. (Webster J. Caye)
Central Concrete Mixing Corp. (A. Johnson)
Cranford, Inc., Frederick L. (F. L. Cranford)
Gallagher, A. C.
Hargen, Stanley
Hoyt, C. R.
Krier, George H.
Lindon, James J.
Moore, Roy Saxton
Piloff, Albert K.
Praeger, Emil
Roux, Maurice G.
Ryan Ready Mixed Concrete Corp. (Joseph J. Ryan)
Bronxville — Westchester County Park Commission (L. G. Holleran)
Buffalo — Bergner, Edward F.
Black Co., John H.
Buffalo Slag Co. (Carlton S. Wickers, Mgr.)
Buffalo Wash Tray Works, (H. A. Blanchard)
Burford, John Nicholas
Cook, F. W.
Cowper Co., John W. (John W. Cowper)
Danaher, W. E.
Degling, A. O.
Diehl, Inc., George C. (Ray P. Diehl)
Egelhoff, R. F.
Federal Concrete Co. (W. E. Jones, Secy.)
Fruchtbaum, J.
Great Lakes Concrete Pipe Co., Inc. (J. G. Miller, Treas.)
Locke, Clyde E.
Monarch Engrg. Co. (H. R. Wait, Pres.)
Muntz, E. P.
Neff, Stewart S.
Stelley, Harry A.
Thompson, Warren H.
*Turner Construction Co.
Ward, Harry A.
Wolf, Robert D.
Carthage — Brownell, Parker J.
Catskill—*North American Cement Corp. (H. F. Kichline, Chief Chemist)
North American Cement Corp. (G. A. White)
Clinton—Clinton Metallic Paint Co. (John Mott, Vice-Pres.)
Cornwall-on-Hudson — Mosher, Clarence H.
Croton - on - Hudson — Schulze, John C.
Floral Park—Pardee, Clarence G.
Glens Falls—*Glens Falls Portland Cement Co. (G. F. Bayle)
*Glens Falls Portland Cement Co. (Geo. F. Bayle, Jr.)
Gloversville — Art Stone Co. (A. C. Suits)
Harmon—Wolpert, Otto
Harrison — Thompson, John H.
Hudson—Weiler, Charles B.
Ithaca—Kuan, F. C.
O'Rourke, Chas. E.
Scofield, H. H.
Urquhart, L. C.
Vanderlip, A. N.
Ward, A. A.
Jackson Heights, L. I.—Halloran, Lt. P. J.
Winiger, Edward
Jamestown — Jamestown Block & Tile Co., Inc., (D. A. McClure, Treas.)
Lackawanna—Buffalo Art Stone Corp. (E. H. DeVoe)
Buffalo Litholite Company (George E. Hipp)
LeRoy — Rib-Stone Concrete Corp. (George E. Priest)
Long Island City—Horn, A. E.
Vago, Oscar J.
Mastic, L. I.—Ferrer, Russell R,
Mt. Vernon — Macneal, LeRoy C.
Newburgh — Heidenreich, Jr., E. Lee
New City—Eldridge, H. W.
New Rochelle— Meltzer, Joseph
Noyes, Haydon T.
Steljes, Martin
New York—Abrams, Prof. Duff A.
Ackerman, Frederick L.
Ambursen Construction Co. Inc. (L. A. Robb)
American Can Co. (C. E. Preis)
American Sika Corp. (Herbert Stoneham)
Anderson, Emanuel
Artstone Products, Inc.
Balcom, H. G.
Barbato, Theodore
*Barney-Ahlers Construction Corp. (John G. Ahlers)
Barney, William Joshua
Barry, L. C.
Barzaghi, Arthur J.
Bell Telephone Laboratories (J. M. Hardesty)
Benedict, James G.
Berg, U. T.
Bertin, René L.
Billner, K. P.
Binney & Smith Co. (J. T. Kealy)
Bird, Millard F.
Bissell, Clinton T.
Boyer, E. D.
Burke, Charles H.
Burrell, H. P.
Chapman, Cloyd M.
Chubb, Joseph
Christensen, Einar
Ciampa, Felix A.
Clos, Charles
Cobb, W. G.
Cohen, A. B.
Columbia University (Albin H. Beyer)
Conahey, George
*Concrete Steel Co. (James F. Curley)
*Concrete Steel Co. (John F. Havemeyer)
Concrete Steel Engineering Co. (Wm. Mueser)
Cooper & Co., Inc., Hugh L. (Hugh L. Cooper)
Cooper, Nelson
Cory, Russell G.
Coyle, David C.
Denton & Co. (P. E. Eisenmenger)
Di Stacio, Joseph
Dixon, D. H.
Dobbs, A. Harry
Dorr Co., Inc., The (W. A. Neill)
Doyle, Frank P.
Durbin, H. R.
Eddystone Cement Co. (E. R. Wilner)
Ellis, Warren L.
Falconer, R. C.
Ferro Building Products Co., Inc. (A. Hamburger)
Fertell, Arthur
Finlay, L. G.
Fischer, L. J.
Floyd, George F.
Fogg, Ralph J.
Fougner, Hermann
Fouilhoux, J. Andre
Foundation Co., The (A. Amson)
Francisco, F. LeRoy
Frankland, F. H.
Fuller & McClintock (Geo. W. Fuller)
Gannett, The J. K.
Gautier, R. C.
Giles, Roy T.
Gilman, Charles
Ginsberg, Frank I.
Glassett, Alfred T.
*Goodwin-Gallagher Sand & Gravel Corp. (John J. Gallagher)
Greenfield, Inc., Arthur (Arthur Greenfield)
Guarantee Constr. Co. (Edward Burns)
Gulick - Henderson Co. (Frank L. Greenfield)
Hadden, Gavin
Hand, Geo. T.
Harris, F. R.
Hay, William Wren
Hayden, R.
Holdsworth Patents Inc. (Harry Holdsworth)
Holz, Herman A.
Holtz & Co. Inc., R. B. (R. B. Holtz)
Hooks, James H.
Hool, George A.
Hunt & Co., Robert W. (J. F. Davis)

Industrial Engineering Co. (D. Traver Miller)
Infante, Percival M.
Integral Waterproofing Co. (Frank J. Phelan)
International Cement Corp. (Howard Rhode)
*International Cement Corp.
*International Cement Corp.
Johnsen, E.
Johns-Manville Corp. (Geo. A. Smith)
Jones, Bevan
Kaufman, David M.
Kinneman, W. P.
Kraus Process Inc. (J. Tyrell Cheney, Pres.)
*Lawrence Portland Cement Co. (E. P. Haubert)
Lawson, David
Lewis Asphalt Engineering Corp. (R. W. Lewis)
Litter, F. J.
McFarland, William G.
McGraw, George B.
McGrew, Jr., E. J.
McNulty, Joseph A.
Manley, Henry
Math, Earle R.
Merriman, Thaddeus
Meyer, Morrison & Co. (B. A. Meyer)
Miner, Joshua L.
Minwax Co., Inc. (Allrich S. Harrison, Vice-Pres.)
Moran & Proctor (G. L. Freeman)
Mueser, William H.
Murray, Inc., Thomas E. (J. R. McColl)
Nailcrete Corp., The (F. B. DeGress)
*Nassau Sand & Gravel Co. (W. J. Timberman)
National Fireproofing Co. (P. Bevier)
Nevin, Thos. M.
Nikitin, P. E.
*North American Cement Corp. (Thos. G. Mooney)
Osborn Co., C. J. (W. F. Purdy)
Parker, Frank S.
Pennsylvania-Dixie Cement Corp. (Blaine S. Smith, Pres.)
Plastic Stone Products Corp. (H. A. R. Zehrlaut, Pres.)
*Raymond Concrete Pile Co. (H. P. Hamlin)
Reichard - Coulston, Inc. (W. A. Kennedy)
Republic Fireproofing Co. Inc. (R. McWilliams)
Reynolds, Frank L.
Rheinstein Constr. Co. Inc. (A. Rheinstein, Pres.)
Richard, N. A.
Richardson, James H.
Roach, M. J.
Rose, Joseph
Rusk, W. Harrell
Rylander, Paul N.
Sanborn, Jas. F.
Sanderson & Porter
Schuster, K. R.
Seabury, George T.
Searl, Thomas D.
Seney, Howard I.
Simpson, Geo.
Smidth & Co., F. L. (O. E. Mogensen, Secy.)
Snare Corp., Frederick (Randall Creamer)
Springer, E. J.
Starrett Bros. & Eken, Inc. (Russell H. Hunter)
Steinman, D. B.
Stephens, A. W.
Stephenson, L. E.
Stern, Eugene W.
Stevens & Wood, Inc. (F. E. Murphy)
Stewart & Co., Inc., James (H. W. Lohmann, Vice-Pres.)
Struckmann, H.
Stuart, Francis Lee
Sundstrom, C. A.
Thompson & Binger (W. D. Binger)
Thompson, James
*Thomson-Starrett Co.
*Toch Bros., Inc. (Maximilian Toch)
Toch Bros., Inc. (Maximilian Toch)
Tomes, P. Austen
Tozzer, A. C.
Turner, Jr., Henry C.
*Turner Construction Co. (E. J. Moore)
*Turner Construction Co. (H. C. Turner)
Underwriters Laboratories (A. R. Small)
*Universal Atlas Cement Co. (E. K. Borchard)
*Universal Atlas Cement Co. (C. R. Hulsant)
Upson, Maxwell M.
Van Buren, Maurice P.
Vandever, Jacob B.
Van Name, R. L.
Voorhees, Gmelin & Walker (Richard A. Backus)
Vulcanite Portland Cement Co. (Albert Meyer)
Warren, Francis B.
Webster, Daniel T.
Weir, C. Leslie
Wheadon, Royntón F.
Whitcraft, L. N.
*White Construction Co., Inc. (G. Edward Escher)
Wigton, C. B.
Wiles, J. R.
Woodard, Silas H.
Wortman, Dietrich
Ytterberg, C. F.
Zipprodt, Roy R.

Queens, L. I.—Furphy, LeRoy F.

Niagara Falls—Wright & Kremers, Inc. (Ernest Kremers)

Nunda—Foote Co., Inc., The (Charles A. Lynch)

Pelham Harbor—Wilcox, Fred C.

Pittsford—Klock, Morgan B.

Rochester — Alexander, Shumway & Utz Co., The (Chas. A. Alexander)
Calif. Stucco Products Co. (H. D. Kneeland)
Campbell, S. A.
Consolidated Materials Corp. (Henry F. Marsh)
Fairchild, LeRoy F.
Norris, J. F.
Rochester, City of, Director of Design and Construc- (Henry L. Howe)
Sanford, L. M.
Scherer, Francis R.
Whitmore, Rauber & Vicinus (L. S. Whitmore, Vice-Pres.)

Rockville Center — Spelman, John R.
Wilson, Francis F.

St. Albans—Mayr, J. T.

Saratoga Springs—Blanchard & Crandell (A. E. Blanchard)

Schenectady — Eighmie, Borden C.
Matlin, W. S.
Union College, C. E. Dept. (Prof. Henry A. Schauffler)

Syracuse—Broga, George F.
Eddy, W. Paul
Eells, W. C.
Paolini, Alfred J.
Solvay Process Co., The
Thomson, Duncan

Troy—Lawson, Thomas R.

Utica—Beebe, Inc., H. R. (H. R. Beebe)

Valley Stream — Smith, George J.

White Plains — Farrell, John J.
Hepburn, William
Solomon, Morton
Weed, Lloyd W.

Woodside, L. I.—Baranzelli Cast-Stone Inc. (G. Baranzelli, Pres.)

Yonkers—Hernandez, Inc. Clinton N. (Clinton N. Hernandez)

North Carolina

Chapel Hill—Hickerson, T. F.

Charlotte—Lee, W. S.
Ornamental Stone Co. (W. J. McCanless, Pres.
Sitton, G. L.

Durham — Duke University Library (Harold C. Bird)
Muirhead Construction Co. Wm. (Wm. Muirhead)

Greensboro—Webb, Stafford

Highland—Sewell, John S.

High Point—Carrick, T. Bright

Hot Springs—Rawls, E. M.

Thomasville—Gray Concrete Co. (F. B. Gray)

North Dakota

Bismarck — Ritterbush Bros. (Robert A. Ritterbush)

Fargo—Fargo Stone & Sand Co. (Joe Ames, Jr.)

Ohio

Akron—Akron, University of (E. F. Schaefer)
Bulger, John

Ashland—Ashland Vault Inc. (H. A. Ledvard, Pres.)

Blue Ash—French, R. M.
Bremen—Stuart, J. H.
Canton—Landor, Edward J.
Cincinnati — Avril Tru-Batch Concrete, Inc. (A. C. Avril, Pres.)
Backman, L. J.
Baldwin, F. G.
*Carey Co., Philip (C. V. R. Fullenwider)
Cincinnati Builders, The (Julius J. Warner)
Cinder Products Inc. (W. Keith Clark)
Dugan Concrete Co.
Evans, L. T.
Euphrat & Hanly (Hunter W. Hanly)
Ferro Concrete Construction Co. (W. P. Anderson)
Ferro Concrete Construction Co. (H. D. Loring)
Garties, George
Goldsmith Metal Lath Co., The (Louis I. Zagoren)
Gould, Harley J.
Hackett, Thos. P.
Hagy, Ernest A.
Hannaford, H. Eldridge
Harding, E. C.
Hodges Construction Co., The (J. E. Hodges)
Johnson, Wm. R.
Morrill, F. W.
Myron, J. A.
Nelson, Norman
Nutting Co., H. C. (C. O. Christy)
Panzer, R. R.
Penker Construction Co., The (Irwin A. Penker, Pres.)
Pollak Steel Co., The (Julian A. Pollack, Vice-Pres.)
*Roos Co., H. W. (H. W. Roos, Pres.)
Roos Co., The H. W. (H. W. Roos)
Roos - Meyer - Hecht Co., The (Geo. W. Meyer, Secy.)
Van Camp Stone Co. (J. H. Liggett)
Waite, H. M.
Ward, William B.
Wetstein, Mentor
Cleveland—Allemann, W. T.
Bolton Pratt Constr. Co. (K. H. Pratt)
Bourne, C. L.
Burger, Alfred A.
Case School of Applied Science (F. H. Neff)
Churchill, H. D.
Daniels, F. W.
Douglas, M. S.
Dykeman, Howard E.
Eberling, C. M.
Emerson Co., The Sam W. (R. E. Laubscher)
Feher, Francis
Forest City Testing Laboratory, The (C. H. Lovejoy)
Grasselli Chemical Co. Inc., The (R. F. Remler)
Hansen, Ralph
Herron Co., The James H. (James H. Herron)
Humphrey, D. S.
Jacoby, H. S.
Johnson, Axel H.
Lambie, J. Edward
Love, Harry J.
Lundoff-Bicknell Co. (C. W. Lundoff)
Master Builders Co., The (S. W. Flesheim)
*Medusa Portland Cement Co. (R. E. Withrow)
Metropolitan Concrete Co. (W. H. Crangle)
Minton Scobell Co. (J. H. Minton)
Morrow, David W.
Osborn Engineering Co., The (K. H. Osborn)
Pease, F. A.
Rackle & Sons Co., The Geo. (Geo. L. Rackle, V. P.)
Rackle & Sons Co., The George (E. E. Rackle, Pres.)
Rawson, R. A.
Rivet-Grip Steel Co., The (H. A. Stuart)
Rogan, W. B.
Scripture, Jr., Edward W.
Sowers, Geo. B.
Stoddard, Ralph P.
Summers, R. E. J.
Thompson, G. Saxton
Thompson, W. N.
Wertz Co., The (Louis S. Wertz, Pres.)
Watson & Associates, Wilbur (W. J. Watson)
Columbus—Anchor Concrete Machinery Co. (G. M. Friel, Mgr.)
Arrow Sand & Gravel Co., The (S. Stepanian, Vice Pres. & Gen. Mgr.)
Blanchard, F. E.
Brown, R. L.
Burgess, Philip
Collins, H. G.
Columbus Testing Laboratories Inc. (A. B. Braden, Pres.)
Concrete Engineering Service Co. (H. C. McCall, Ch. Engr.)
Cunard Lang Concrete Co. (L. D. Hagerty)
Elford, H.
Hindman, W. S.
Lakewood Engineering Co. (Lion Gardiner, Vice-Pres.)
Large, George E.
Morris, Clyde P.
Shank, J. R.
Dayton—Clement, R. B.
Condit, G. H.
Crume Brick Co., The (W. H. Crume)
Schad, Bernard T.
Smith, Oscar S.
East Liverpool—Frosch, A. E.
Evanston—Fisher-DeVore Construction Co. (Frank F. Fisher)
Greenville — American Aggregates Corp. (Guy C. Baker)
Hamilton — Hamilton Gravel Co. (W. P. Watson, Secy-Treas.)
Kingston — McBeth, R. W.
Lakewood — Deinboll, F. K.
Feldrappe, M. G.
Withrow, Roe E.
Lisbon—Thomas & Sons Co., The R. (H. P. Sleeman)
Newark—Kerr, George W.
Wyeth-Scott Co., The (W. D. Wyeth, Pres.)
Osborn—Jennings, Wm. A.
Painesville — Standard Portland Cement Co. (Erie J. Ochs, Supt.)
Portsmouth — Goodman, Hal W.
Silica—Sleight, T. R.
South Park — Hydraulic Press Brick Co.
Toledo—Bates, Frederic G.
Blanchard, Arthur H.
Champe, Geo.
Finishing Lime Assoc. of Ohio, The (L. E. Johnson)
Hausman, Steel Co., The
Kriege, Herbert F.
Kuhlman Builders Supply & Brick Co., The (C. E. Kuhlman)
McKee, S. C.
National Cement Block & Supply Co., The (C. B. Fellabaum)
Rhines, Geo. V.
Spieker Co., The Henry J. (W. E. Condit)
Toledo Concrete Pipe Co., (L. J. Loughlin)
Warren—Harm, George O.
Youngstown—Applegate, L. M.
Gorsuch, V. D.
Hubbard, Fred
Irwin, Orlando W.
Nixon, H. O.
Roscoe, Ralph E.
Snodgrass, R. D.
Standard Slag Co. (F. E. Miller)
*Truscon Steel Co. (Julius Kahn, Pres.)
Youngstown Burial Vault Co. (E. E. Kling)

Oklahoma

Ada—Bayless, O. A.
Duncan — Halliburton Oil Well Cementing Co. (Hayden Roberts, Chemist)
Holdenville—Votaw, Curtis H.
Oklahoma City—Brown & Myers
Clark, B. E.
Eckert, Ralph T.
Harter-Marblecrete Stone Co. (B. D. Harter)
Oklahoma Testing Laboratories (Wm. Furber Smith)
Verity, Fred L.
Stillwater—Lothers, John E.
Oklahoma A. & M. College, (Mrs. Elsie D. Hand, Librn.)
Saxton, Ren G.

Tulsa—Oklahoma Glazed Cement Pipe Co. (J. N. Chandler, Pres.)

Oregon

Albany—Harber, G. Peyton
Corvallis—Glenn, B.
Waterman, Ivan F.
Owyhee—Betts, Clifford A.
Portland—Alin, A. L.
Claussen & Claussen, Inc. (W. E. Claussen, Secy-Treas.)
Cooper, Miles K.
Hammond, Inc., Ross B. (Ross B. Hammond)
Henny, D. C.
Nichol, F. E.
Nims, Chas. B.
Standard Art Stone Co. (T. J. Schmidt)
Thomas Cast Stone Co., Ernest (Ernest Thomas, Pres.)
Salem—Paxson, C. S.

Pennsylvania

Aliquippa—Stetson, F. M.
Allentown — Ashton, Ernest
Brickett, Edward M.
Cement-Gun Co., Inc., (B. C. Callier)
Hollywood Building Block Co. (Paul R. Klotz)
*Lehigh Portland Cement Co.
Lehigh Portland Cement Co. (E. M. Young, Pres.)
*Lehigh Portland Cement Co. (E. M. Young, Pres.)
Lehigh Portland Cement Co.
McCready, E. F.
Meckley, E. W.
Oscar, L. C.
Pearson, J. C.
Watt, A. G.
Young, Roy N.
Ambler—Asbestos Shingle, Slate & Sheathing Co. (Richard V. Mattison)
Bethlehem — Kreidler, Carl L.
Lyse, Inge
Parkinson, G. W.
Slater, Willis A.
Sutherland, Hale
Bryn Mawr — Vauclain, Jacques L.
Butler—West Penn Cement Co. (O. J. Binford)
Carlisle—Horner, J. M.
Catasauqua — Allentown Portland Cement Co. (R. S. Weaver)
Fuller Co. (P. F. Stauffer, Sales Mgr.)
Cementon—Hoke, Arnold
Center Square—Young, Jr., Fred W.
Columbia—Rule, H. Paul
Easton—*Alpha Portland Cement Co. (Louis Anderson, Jr.)
*Alpha Portland Cement Co. (F. G. McKelvy)
Bragg, J. G.
Dewson, S. H.
Messinger Supply Co. (Dr. Wm. Mock, Secy.-Treas.)
Williams & Co., C. K. (E. G. Davies, Vice-Pres.)
Erie—Erie Patent Block Co.
Johnson, W. P.
Nickel Plate Sand & Gravel Co. (H. F. Rath, Sec.)
*Shenk Co., Henry (E. R. Shenk)
Wadsworth, John Frederick
Frankford—Button, William G.
Franklin—*Franklin Steel Works (E. E. Hughes)
Gettysburg—Clutz, Frank H.
Glenside — Albright & Mebus (Charles F. Mebus)
Friel, Francis S.
North Glenside—Doane, Louis H.
Eitzen, Henry R.
Harrisburg—Masters, F. M.
Pennsylvania State Highway Dept. (W. H. Connell)
Ulman, Malcolm H.
Kingston — Nepenna Building Materials Co. (F. L. Schott)
Lancaster — Krewatch, Wm.
Lancaster Concrete Tile Co. (Henry Boettcher, Pres.)
Malone, John A.
Wickersham, John H.
Lansdowne — Kirk, Joe Neil
Mahanoy—Jones, William A.
Marcus Hook — Delco Concrete Products Co. (G. L. Crosgrove, Mgr.)
Milmont Park—Denuel, J. M.
Mount Union—Andrews, E. T.
Moylan—Radbill, Ray
Nazareth—Champion, E. C.
*Hercules Cement Corp.
*Nazareth Portland Cement Co.
Robeson, Harold B.
New Castle—Melvin, Russell C.
New Kensington—Fletcher, B. J.
*Straub, F. J.
Norristown—Long, James B.
Northampton — Moyer, Wm. N.
Northmont - Reading — Berks Building Block Co. (G. M. Muth, Treas. & Mgr.)
Palmerton—New Jersey Zinc Co.
Penn Valley — Corridon, J. B.
Philadelphia—Baily, R. W.
Ballinger Co., The (Jean Paul Rickter)
Baylson, J. Jacob
Berry, H. C.
Bowman, H. L.
Brennan, Frank X.
Brown, Paul G.
Bruner, Louis S.
Campion, H. T.
Cassell, J. D.
Coffman, Herbert
Concrete Protection & Color Co. (John McKay)
Conwell & Co., E. L. (E. L. Conwell, Pres.)
Cooper, W. R.
Covell Corp., The
Delaplaine, Henry
Drehmann Paving Constr. Co. (C. E. Drehmann)
Formigli Architectural Stone Co. (Paul Formigli, Sec.)
Foster, Jr., Alexander
Gaston, H. F.
*Giant Portland Cement Co. (Chas. F. Conn, Pres.)
Gibson, W. Herbert
Glasheen, R. E.
Gravell, Inc., Wm. H. (Wm. H. Gravell, Pres.
Hallahan, John P.
Hebold, Denis O.
Hibbs, Manton E.
Hilts, H. E.
Hipwell, Howard S.
Horst Co., Henry W. (A. E. Horst)
Johnson, Virgil L.
Knopel, Herbert J.
Lamb Co., Robert E. (Robert E. Lamb)
Lesley, Robert W.
Lindsay & Co., Inc., W. W. (James C. Newlin, V. P.)
*Lone Star Cement Co. (H. E. Hilts)
Louchheim, Wm. S.
McComas, Wm. E.
Mauro, Francesco
Mueller, Harold P.
*National Building Units Corp.
Nelson Pedley Construction Co. (Joseph Milgram)
Peiper, A. S.
Perrot, Emile G.
Power & Son, Eugene S. (E. S. Power)
Reynvaan, A. J.
Ridge, R. S.
Riehle Bros. Testing Machine Co. (W. C. Moran)
Rogers Co., J. S. (J. Clyde Rogers)
Roth, Jr., Charles Oliver
Sauter, Wm. R.
Seward, Harold C.
Schautz, C. Paul
Sinclair & Grigg (Harris Britt)
*Steele & Sons Co., Wm.
Steele & Sons Co., Wm.
Stevens, Charles H.
Stewart, Frank G.
Tatnall, Francis G.
Thompson, Carroll R.
Thorn, Jr., Frederick G.
Timlin, M. A.
*Turner Construction Co. (J. A. Turner)
Uhr, Saul

*United Engineers & Constructors Inc. (Prof. F. O. Dufour)
Van Asdlen Construction Co. (W. S. Van Asdlen)
Wagner, Samuel T.
Warner Company
Warren Company, F. V. (Lewis Tower)
Watson, T. P.
Webb, Walter Loring
White & Co., Barclay (Barclay White)
*Whitehall Cement Mfg. Co. (P. D. Gaskill)
Williams, Timothy S.
Wilson, A. R.
Winslow Construction Co. (John S. Krauss)
Witmer, Francis P.
Woodle, Jr., A. S.
Wyatt, W. Kirk
Pittsburgh — Aluminum Company of America (James W. Rickey)
American Reduction Co. (Jay Moore)
Anderegg, F. O.
Benedict, E. L.
*Blaw-Knox Co.
Burgess, C. C.
Burke, J. E.
Burks, I. E.
Campbell, John T.
Casey Co., John F. (Samuel Leslie Fuller)
Concrete Products Co. of America (C. F. Buente)
Connar, V. N.
Dann, Alex W.
Dibert, Grant
Dunnells, Clifford G.
Duquesne Slag Products Co. (C. L. McKenzie)
Freeman P. J.
General Cement Products Co. (R. D. Campbell)
Haggert, C. N.
Hayner, J. L.
Imlay, R. G.
Jones & Laughlin Steel Corp. (J. B. Carlock)
Kaiser, B. J.
Kubitz, Fred
Lambie, Joseph S.
Larkin, Chas. W.
Levison, Arthur A.
McCullough, Frank M .
Metzger - Richardson Co. (F. L. Metzger)
Meyer, B. F.
Mylrea, T. D.
National Steel Fabric Co. (W. H. Shaffer, Jr.)
Norris, W. H.
Pittsburgh, City of (Winters Haydock)
Pittsburgh Testing Laboratory (A. R. Ellis, Gen. Mgr.)
Plagwit, Eric
Prack, Bernard H.
Price, Philip W.
*Rust, H. B.
Saurbrey, Alexis H.
Standard Inspection Co. (John M. Bailey)
Stotz, Jr., Edward
Vegeler, W. F.
Western Pennsylvania Sand & Gravel Assn. (Ray V. Warren)
Westinghouse Electric Mfg. Co. (J. E. Webster)
Westinghouse Electric & Mfg. Co. (Dr. H. C. Harrison)
Wilcox, Inc., Ira C. (Jno. Rishel)
Wimer, D. C.
Wiseman, W. J.
Pottsville — Pottsville Building Block Co. (Barton R. Biever)
Reading—Hoffman, A. E.
Muhlenberg, Charles R.
Roxborough — Flanagan, Charles A.
Safe Harbor—Robb, Chas. G.
Scranton—Baker, Samuel
Smoley, Constantine K.
Sharon—Bishop, Howard
State College—Minshall, R. E.
Swarthmore—Lilly, Scott B.
Swissvale—Abel, Norman A.
Upper Darby — Artcrete Products Co. (Clarence E. Kuemmerle)
West Manayunk—Painter, Geo. M.
Wilkes-Barre — Luzerne County Road & Bridge Dept. (Robert L. Williams)
Rosser, Robert C.
Wilkinsburg—Frick, Walter H.
Hoffman Co., F. (Wm. F. Hoffman)
Williamsport — Williams, E. C.
Wilmerding — Livermore, A. D.

Rhode Island

Cranston—Steimer, William A.
Pawtucket — Builders Concrete Stone Co. (Fred D. Fuller)
Providence—Ballou, Henry W.
Benson, Newton D.
Corrado, Anthony
Holton, Jr., Philip J.
Maguire Co., Chas. B.
Merchant, Archie W.
Nolan, S. Frank
Providence Transit Concrete Corp.
R. I. State Bd. of Public Roads (J. V. Keily)
Sheldon & Son, F. P.
Swanson, Inc., G. Fred (G. Fred Swanson, Pres. & Treas.)
Watt, Edwin L.
Shawomet—Bragger, E. Y.

South Carolina

Anderson—Guest, Walter C.
Columbia—South Carolina State Highway Dept. (N. S. Anderson, Maint. Engr.)
South Carolina Highway Dept. (W. H. Mills, Jr., Test. Engr.)
Beaufort, Sea Islands—Gleason, Kate

South Dakota

Canton—Canton Block & Tile Co. (E. D. Rowe)
Rapid City—South Dakota Cement Plant (William Fowden, Supt.)
Vermillion—Pletta, Dan H.
Watertown — South Dakota Concrete Products Co. (Hugh H. Johnson, Mgr.)

Tennessee

Alcoa—Nowlin, Wm. D.
Chattanooga—Klein, W. H.
Pennsylvania-Dixie Cement Corp. (E. P. Newhard)
*Signal Mountain Portland Cement Co. (John L. Senior, Pres.)
Smallwood, L. C.
Cowan—Ernst, Walter S.
Fountain City—Nielsen, S. W.
Jackson—Yandell & Conger (C. H. Yandell)
Johnson City—Brekke, G. N.
Knoxville — McCroskey, Thos.
Ready Mixed Concrete Co., John L. Humberd, Pres.)
Southern Cement Products Co. (Fred W. Weigel)
Lebanon—Gentry, J. W.
Memphis—Allen, Thos. H.
Busse, F. A.
Hollywood Concrete Pipe Co. (O. H. Miller, Pres.)
Howe, H. N.
Hunter, Harry B.
Loeb, II, Henry
Nichols, C. R.
Schevenell, V. E.
Sullivan, F. Daly
Nashville—Bauman, E. W.
Burr, Henry A.
Coolidge, W. A.
Franklin Limestone Co. (A. B. Rodes)
Gould Contracting Co. (C. B. Wilson)
Hansard, Orren H.
Hopton, H. V.
Nashville Breeko Block & Tile Co. (Chas. Wm. Akers, Mgr.)
Norred, Jr., C. V.
Rowan, Wm. H.
Wilson-Weesner-Wilkinson Co. (W. C. Sensing)
Richard City—Shuman, E. C.

Texas

Austin — Ferguson, Phil. M.
Giesecke & Harris (Munsey Wilson)
University of Texas, Dept. of Civil Engineering (Dean T. U. Taylor)
University of Texas, Engineering Experiment Station (Raymond F. Dawson)
Wright, Chester C.

Brownwood—Keesee, Gerald B.
Bryan—Butler, R. B.
College Station — Texas Engineering Experiment Station (F. E. Giesecke, Director)
Richey, Prof. J. J.
Dallas—Browne, E. F.
Clapper, Lyle
Clark, Charles A.
Ferguson, Lewis R.
Gill, Grayson
Jameson, R. O.
Knox, Jean H.
*Lone Star Cement Co. (L. R. Ferguson)
Nash, Jack T.
Penniman Concrete & Material Co. (Nolan Browne, V. P.
Reaney, Charles F.
Simpson, C. F.
Watson Company (G. W Andersen)
El Paso—Anderson Bros. (W. R. Anderson)
Deedy, Walter E.
McKee, Robert E.
Southwestern Portland Cement Co. (Wm. R. Blair)
Trost, A. G.
Fort Worth—Byrne, Thos. S.
Gillette, Harold S.
Southwestern Laboratories, (F. B. Porter, Pres.)
Freeport—Burns, Homer S.
Rhodes, J. Morton
Galveston —Fraser, Andrew
Houston — Calhoun, Jr., John C.
Houston, City of (H. L. Show, City Engr.)
Dannenbaum, J. B.
Fugate, G. L.
Gottlieb, R. D.
Gulf Concrete Pipe Co. (H. E. Johnsen, Vice-Pres.)
McKenzie, J. G.
Weismann, George F.
Laredo—Zachry, H. B.
Lubbock—Haynes, S. B.
Peters, Strange & Bradshaw (N. L. Peters)
Texas Technological College (J. H. Murdough)
New Braunfels—Dittlinger Lime Co. (E. Eikel)
Pharr—Devine, P. S.
San Antonio — Beretta Engineers, Inc., J. W. (J. W. Beretta, Pres.)
Diver, M. L.
*San Antonio Portland Cement Co. (C. Baumberger, Pres.)
Simpson Co., W. E. (W. E. Simpson)
Southern Cement Products Co. (Harvey Perrin)
Victoria—Killian, M. B.
Waco—Central Texas Iron Works (L. H. Wood)
Johnson, O. K.

Vermont

Bennington—Burrington, Jr., M. J.

Virginia

Blacksburg—Begg, R. B. H.
Bristol—Stone, G. M. & G. C.
Clarendon — Mitchell, Nolan D.
Henry—Blue Ridge Talc Co., Inc. (C. O. Kitson, Secy.-Treas.)
Hopewell—Rowland, Walter
Norfolk—*Lone Star Cement Co. Va., Inc. (Dwight Morgan)
Onley—Norfolk, Parker F.
Richmond — Economy Concrete Co. of Va., Inc. (Ernest Wiedemann)
Froehling & Robertson (T. B. Hartless)
Lee, Smith & Van Dervoort, (Jameson Van Dervoort)
Macatee, W. R.
Thurston Co., Inc., W. P. (W. P. Thurston)
Riverton—Riverton Lime Co. (G. H. Hutchinson)
Roanoke—Ferguson, M. W.
South Washington — Davis, Herbert A.

Washington

Bellingham — Olympic Portland Cement Co. Ltd., The (A.F. Krabbe, Gen. Supt.)
Bremerton—Jack, Eugene C.
Morreell. Ben
Concrete—Superior Portland Cement Co. (C. L. Wagner)
Everett—Everett Concrete Products Co. (Hans Mumm, Jr.)
Olympia — Washington State Highway Dept. (Bailey Tremper)
Williams, Chas. H.
Port Angeles — Owen, Thos. W.
Puget Sound—Cotter, Lt. Comdr. C. H.
Seattle—Boyle, Ernest E.
Bushnell, H. L.
Cavanagh, Jr., W. Curran
Collier, Ira L.
Faulkner, H. F.
Gleason, W. A.
Hadley, H. M.
Hitchings, A.
Hull, Herbert F.
Hunt, Thomas D.
Hutton, S. E.
Lincoln, R. W.
More. Chas. C.
Northwest Concrete Products Assn. (F. R. Zaugg)
Northwest Steel Rolling Mills, Inc. (Paul H. Steuding, Sales Mgr.)
Ober, R. H.
Olympian Stone Co. (Leo Swartz)
*Olympic Portland Cement Co. (W. P. Cameron)
Pacific Coast Cement Co. (Ned H. Nelson)
Pacific Coast Steel Corp. (W. A. Roth)
Pacific Stone Co., The (Gilbert E. Tucker)
Perry, L. A.
Pioneer Sand & Gravel Co. (Phillip Morrison)
Seattle, City of
Seattle, City Engineer's Dept.
Sheffield, F. D.
Stevenson, J. H.
Strandberg, E. L.
Suckow, V. C.
*Superior Portland Cement Co. (E. P. Lucas)
Sylliaasen, M. O.
Trueblood, Paul M.
Uhrich, M. J.
Vaso Manufacturing Co. (Carl L. Leon, Mgr.)
Ward, Inc., J. F. (J. F. Ward)
Way, W. F.
West, J. R.
Winston Bros. Co. (W. H. Gardiner)
Witte, Herman Calvert
Tacoma—McKay, W. S.
Smallwood, H. S.
Yakima—Yakima Cement Products Co. (W. P. Hews)

West Virginia

Alderson—McThenia, A. W.
Huntington — Schonthal, D. C.
Martinsburg—Young, W. J.
Morgantown—Brooks, W. E.
Davis, Fred A.
Wheeling—Faris, Frederic

Wisconsin

Appleton—Koepke Bros. Construction Co. (G. A. Koepke)
Fond du Lac—Hutter Construction Co. (George F. Hutter)
Immel Construction Co. (Harry W. Mabie, Jr.)
Johnson, Robert C.
Kenosha — North Shore Cement Burial Vault Co. (Hubert McQuestion)
Madison—Blue Jay Concrete Products Co. (Chas. Clare)
Fritz Co., C. B. (N. J. Schmitz)
Larson, Guy H.
Madison Silo Co. (C. C. Woody)
Wisconsin University Library (W. M. Smith)
Withey, Morton O.
Manitowoc — Manitowoc Portland Cement Co.
*Manitowoc Portland Cement Co. (E. J. Maguire, Vice-Pres. & Treas.

Milwaukee — Browne, Walter E.
Burmeister, R. A.
*Chain Belt Co. (B. F. Devine)
Christoffel Art Stone Co. (John Christoffel)
Cincrete Corp. (L. E. Pitner)
Clicquennoi, I. M.
Coddington Engrg. Corp. (Samuel C. Coddington)
Consumers Supply (S. N. Brown)
Devos Co., Inc., Wm. H. (A. W. Devos)
Eagle Chemical Co. (Chas. Schrank)
Foster Construction Co., The (Harry L. Foster)
Franklin, Jack
Froemming Brothers Inc., (B. A. Froemming)
Hahn, Harold S.
Harris, Wallace R.
Hirschberg, Walter P.
Hirth Concrete Breaking Co. (G. W. Hirth)
*Koehring Company, The
*Koehring Company, The (E. H. Lichtenberg)
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Larson, Louis J.
McKown, G. W.
Milwaukee Concrete Products Cooperative Assoc.
*National Equipment Corp., The
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Ricketson Mineral Color Works (Frederick C. Bogk, Pres.)
Riesen's Sons Co., Paul (Louis F. Reuter, Jr.
Rose, T. L.
Schmitt & Son, Inc., H. (Eugene H. Schmitt)
Siesel Co., S. M. (S. M. Siesel, Pres.)
Simmond, Fred C.
Stanage, John L.
Talbot, Kenneth H.
Tubesing, William F.
Whitney, Chas. S.
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Racine—Julius Sorenson & Sons
Sheboygan — Donohue, Jerry
Superior—Buck, R. C.
Waukesha — Butler Bin Co. (Morgan R. Butler, Pres.)
Wausau — Wausau Concrete Co. (C. W. Parsons, Pres.)

Alaska

Juneau—Krause, G. E.

Canal Zone

Ancon—Bunker, Geo. C.
Balboa Heights—Hertz, A. L.
Jones, R. C.
Moore, L. B.
Randolph, Edward Sydney

Panama

Panama City — Arango, Henrique G.
Hidalgo, Rafael A.
Puerto Armuelles — Millen, Vincent

Hawaiian Islands

Hilo—Will, Charles H.
Honolulu—Bishop Estate, B. P. (Theo. B. Bush)
Hawaiian Contracting Co. (H. P. Benson)
Miller, G. R.
Morgan, J. Grant
Robert, E. E.
Pearl Harbor—Manning, J. J.

Philippine Islands

Manila—Siochi, Pedro
Licuanan, Jose G.

Porto Rico

Guayama—Lucchetti, A.
Mayaguez—College of Engineering (Frank O. Kerr)
Moto, Candelario Calor
San Juan—Benitez & Benitez Gautier
Gonzalez, Adriano
Dept. of the Interior (Commissioner Guillermo Esteves)
Totti, Etienne
Santurce—Gonzalez, Carl E.

Canada

Alberta

Calgary—Bennett, John G.
Lain, John Sayers-Neil, J. S.

British Columbia

Vancouver—Barclay, J. B.
British Columbia Electric Ry. Co. Ltd. (E. E. Carpenter)
Brown, Philip P.
Grant, J. R.
Hamilton, Chas. T.
Mackney, A. E.
Marble, William O.
Northern Construction Co. Ltd. (Wm. Smaill)
Victoria—*British Columbia Cement Co., Ltd. (Edwin Tomlin, Director-Treas.)

Manitoba

Brandon — Fulcher, Edmund
Winnipeg—Cowin & Co. Ltd. (H. B. Henderson)
Greater Winnipeg Water District (W. M. Scott)
Library, Univ. of Manitoba
National Testing Laboratories, Ltd., The (L. J. Street)
Peterson, F.
Rensaa, E. M.

New Brunswick

Saint John—Evans, Edwin Ronald
Johnston, W. J.
McKinney, J. Harold

Nova Scotia

Halifax—Sexton, F. H.
Noonan, W. H.
Nova Scotia Power Commission, The (Harold S. Johnston)

Ontario

Campbellford — Burgar, F. A.
Cochrane—McCarthy, T. V.
Fitzroy Harbor—Mackie, Thos. W.
Morrow & Beatty Ltd. (J. A. Beatty, V. P.)
Ford—Merlo, Merlo & Ray, Ltd. (Louis Alvin Merlo, Pres.)
Hamilton—Brown, John A. W.
Canada Crushed Stone Corp. (C. M. Doolittle)
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Johns, H. S.
Piggott, Joseph M.
Kemptville — Dominion Concrete Co., Ltd., The (W. G. Anderson)
Dominion Concrete Co., Ltd., The (W. G. Anderson, Pres.)
Ottawa — Canadian Engineering Standards Assn. (B. Stuart McKenzie, Sec.)
Canadian National Parks Branch (J. B. Harkin)
Hayley, Harry
Lucas, J. W.
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Peerless Stone Ltd. (F. E. Beer)
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Walkerville — Trussed Concrete Steel Co., of Canada, Ltd. (E. G. Ryley, Chief Engr.)
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Quebec

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Quebec Dept. Pub. Wks. & Labor (Ivan E. Vallee)
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Australia

Adelaide—James, W. H.
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Geelong—Australian Cement, Ltd. (W. B. McCann, Gen. Mgr.)
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Maryborough — Jorss, Chris A.
Melbourne — Concrete Construction Pty. Ltd. (L. A. Deegan)
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Commonwealth Portland Cement Co. (Fred J. Beardmore, Secy.)
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Hinder, R. B.
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Main Roads Board of N. S. W. (T. H. Upton)
McLean, William King
Rankin, John
Standards Association of Australia (W. R. Hebblewhite)
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Woollahra—H. G. Verey

Belgium

Antwerp—Cimentaries Et Briquetaries Reunies C. B. R. (F. Van Ortroy, Secy.)
Brussells — Groupement Prof. des Fabricants de Cement Portland Artificial de Belgium (G. Deroover)
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Liege—Franki, Pieux

Brazil

Campos—Vieira, E. de Moraes
Rio de Janeiro — Baumann, Alfred
Baumann, Herman
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Carvalho, Manfredo de A.
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Harboe, Helge
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Reis, F. Santos
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Alves de Lima, Jorge
Fonseca, Humberto da
Filho, Julio Miguel Freitas
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Urner, J. P.

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Sandakan—Bishop, Harry

Chile

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Valparaiso — Claussen, Carlos F.
Vina del Mar—Feuereisen, Eduardo H.

China

Canton—Bjuke, Capt. N. Rob
Dairen — Onodo Cement Co. Ltd. (K. Mizoguchi)
Mukden—Kao, P. K.
Shanghai—Lee, Kung Wu, Pond S.
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Pei Yang University Yu, M. T.

Cuba

Havana—Cristofol, J.
Fitz Gerald, C. C.
Martin, Juan San
Martinez, Jose Ricardo
Rodrigvez, Lvis M.
Vila, Jose Antonio
Santiago—Formell, Victor M.
O'Fallon, R. S.

Czechoslovakia

Bratislava—Franc, Joseph

Denmark

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Egypt

Alexandrie — Gazaleh, Edouard
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England

Bristol — Robertson, Andrew
East Molesey — Smith, Major N. H.
Garston—Glanville, W. H. Stradling, R. E.
Halewood—Burgess, L. T.
Harrogate—Button, Harold
Ness, Gilbert
Haverton Hill—Kimpton, A. C.
Hull—Earle, Ltd., G. & T. (G. F. Earle, Mgr. Dir.)
Ipswich—Tetsall, E. P.
Leeds—Airey, Sir Edwin
Leicester—Ellis & Sons, Ltd., John (Basil C. Sharman)
Liverpool—Bates, Henry K.
London—Ames, John
Associated Portland Cement Mfrs., Ltd., (H. R. Cox)
Barron, S. N.
Brabant, Harry M.
Butler, D. B.
Cast Concrete Products Assn. Ltd., The (A. S. Windsor)
Empire Stone Co., Ltd.
Garrett, W. S.
Hambley, Percy Noel
Mouritz, C.
Osler, Richard H.
Props of Hay's Wharf Ltd., (H. J. M. Skidmore)
Riley, Harbord & Law (E. W. Palmer)
Smith, R. A. B.
Stanger, R. H. Harry
Stewart, Wm. J.
Stuart's Granolithic Co. Ltd. (Wm. Adams, Mgr. Director)
Wallace, W. K.
Manchester — Bentham Cecil
Bridges, G. P.
Singleton-Green, J.
Stokes, G. W.
Plymouth—Blake, F.
Rugby—Blyth, Charles E.
Brooks, Ernest W.
Kaye, Lister Lister
Scunthorpe — Hydroprest Products Co. (A. Ogilvie)
Sheffield—Turner, Frank C.
Stockport — Mercer, L. Boyd
Wednesbury—Wilson, N. W.
Wolverhapmton — Cotterell, Ernest L.
Yorks—Marsden, R. W.

Finland

Helsingfors — Cement-producenternas (Urho Aberg)
Osakeyhtio Constructor Aktiebolag
Pargas—Pargas Kalkbergs Aktiebolag
Vaasa—Waasan Rautabetoni O. Y. (A. Lepisto)

France

Grasse—Magny, A. V.
Le Teil — Ste. Amedes Chaux et Ciments de Lafarge et du Teil (M. Rengade)
Paris—Hendrickx, Jean
Societe Des Ciments Francais (F. Turquais)
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Germany

Berlin—Burchartz, H.
Haegermann, Dr.
Klank, Willy
Cologne—Stierlen, Wilhelm
Weerdt, H. E. de
Darmstadt — Kleinlogel, Prof. Adolf
Dusseldorf—Grun, Richard
Hover—Verein Deutscher Portland-Cement-Fabrikanten (Dr. Rudolf Kneisel)
Karlsruhe — Probst, Dr. Emil
Obercassel — Huser, Dr. Ing. Alfred
Oppeln—Simon, Dr. Nikolaus
Stuttgart — Graf, Prof. Otto

Holland

Amsterdam — Dekker, F. W.
De Steeg—Vigorose Cement Industrie, N. V. (B. A. Honig)
Delft—Technische Hoogeschool (J. A. Bakker)
The Hague—Dekker, P.
Ringers, J. A.

Honduras

La Ceiba—Suter, Oscar M.

India

Alwar City—Kohli, T. N.
Attock—Punjab Portland Cement, Ltd.
Barot—Sarwar, M. G.
Bengal—Nag, N. K.
Colaba—Baingolkar, H. V.
Bombay—Chhapgar, F. K.
Concrete Association of India (Alan Moncrieff)
Daruvala, J. P.
East, Frederick A. H.
Engineer, R. K.
Hartley, D. R. C.
Indian Hume Pipe Co. Ltd., The
Joglekar, M. V.
Kamat, V. A.
Master, R. S.
Noel-Paton, V. F.
Setna, J. N.
Calcutta—Choudhury, A. C.
Kynnersley, T. R. S.
Sanyal, A.
Delhi—Bose, N. N.
Israil, Mohammad
Siddique, Bashie Ahmad
Karachi — Bhaledino, Kambarali
Lakheri—Bundi Portland Cement, Ltd. (E. Christensen, Mgr.)
Christensen, E.
Larkana—Badlani, N. R.
Dunsdon, A. C.
Madras—Gray, T. Campbell
Meerut — Sharma, Hari Shankar
Mirpurkhas — Gazder, Mahamed Hashim
Mysore—Kikkeri, S. A.
Nasik—Thomas, A. Rylands

Ireland

Belfast—McIlmoyle, R. L.
Cork—Walsh, Prof. H. N.

Italy

Bergamo—"Italcementi" (Carlo Vigliani)
Cagliari—Imprese Idrauliche Elettriche
Cosenza—Steiner, Angelo
Genoa — Ing. Mantelli, Reg. Corbella & Co. (I. Mantelli)
Milano — Laboratorio Prove Materiali, Della R. Scuola Di Ingegneria (Giulio Revere)
Loro & Parisini
Ruegg, W. O.
Soc. Anonima Imprese Generali
Vandone, Italo
Roma—Associazione Italiana per gli Studi sui Materiali da Construzione (Camillo Guidi)
Torino — Albenga, Prof. Giuseppe
Pozzo, Alberto
Regia Scuola D'Ingegneria, Biblioteca
Regia Schola D'Ingegneria di Pisa (Prof. Giovanni Quaglia)
Soc. Porcheddu Ing. G. A. (G. A. Porcheddu)

Japan

Chichibu—Ohtomo, Kosuke
Fukushimaken — Komuro, Mangoro
Kyoto—Ban, S.
Kondow, Yasuo
Seoul—Okubo, Toshiyuki
Tokyo—Abe, Mikishi
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Naito, Tachu
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Soga, Susumu
Suzuki, Hajime
Uchimura, Sabro
Yamashita, Toshirow
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Mexico

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Ornelas, Samuel
Orozco, J. Vicente
Weiss. Andrew
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Cuevas, Jose A.
Garita, Ricardo Cicero
Gomez-Perez, Francisco
Howell, C. H.
"La Tolteca" Cia de Cemento Portland (G. H. E. Vivian)
Rebolledo, Miguel
Blank, Alton J.
Puebla—Blank, Alton J.
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New Zealand

Auckland — Dibble, S. Trever
Jeffrey, W. McKenzie
Murray, Andrew
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Wilson's Portland Cement, Ltd.
Dannevirki—Gillon, Gerald Norton
Nelson—Jack, Erick D.
Dunedin—Milburn Lime & Cement Co., Ltd. (J. H. Stewart, Gen. Mgr.)
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Palmerston North — Wright, A. H. W.
Wanganui — Hodge, Ernest R.
Wellington — Anscombe, Edmund
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Norway

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Br. East Africa

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South Africa

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Straits Settlements

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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE, VOLUME 27

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Surface treatment" and other subjects classified under this head, are indented.

Key words to important subjects appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Blast furnace slag," each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names appear in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to "ENGINEERING DESIGN" or "TESTS OF MEMBERS AND COMPLETED STRUCTURES" thus avoiding an oversight by the searcher of important allied data.

The readiest use may be made of this index by gaining some familiarity with the thirty main classifications.

The searcher for information on design and proportioning of concrete mixtures will look for MIXTURES and refer to subheads; "Design" and "Proportions." Field control methods will be found under TESTS AND FIELD CONTROL.

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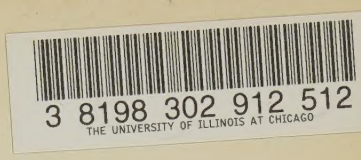
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ERRATA—ABSTRACTS

Nomogram for calculating allowable distance between abutments of vertical or inclined reinforced concrete slabs as determined by diagonal tension and shear.

P. 146, lines 26-32 should be as stated in the author's correction, p. 304, lines 30-36.

Experiences from a power plant construction in India, p. 180, line 29, *The Swedish contracting company "Vatten byggnadsbyran"* should be *The Swedish consulting engineers Vattenbyggnadsbyran.* The contractors of the civil engineering work at the power station were Messrs. Topham, Jones and Railton of London.


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